SEP 28 1959

PRESTRESSED CONCRETE INSTITUTE

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JOURNAL

OF THE PRESTRESSED CONCRETE INSTITUTE

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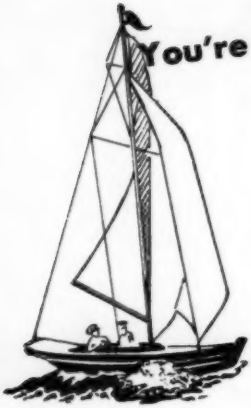

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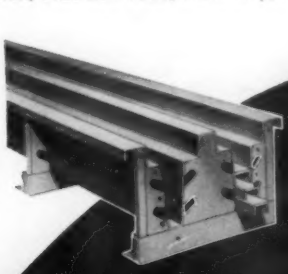

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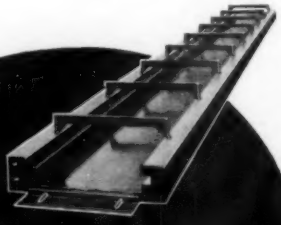

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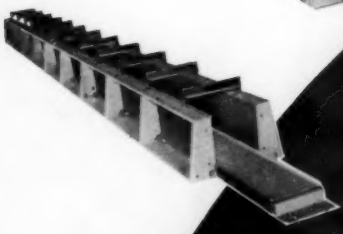

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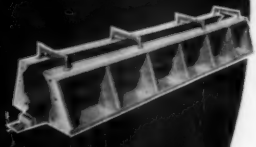

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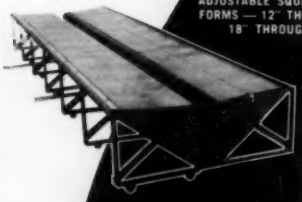

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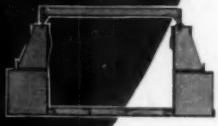

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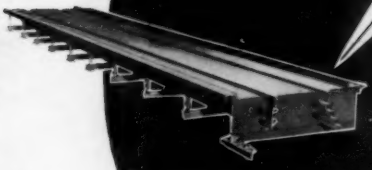

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LARGEST OF 6 BRIDGES includes five 120-foot spans of prestressed POZZOLITH concrete beams. Bridge deck is 71-feet wide. Architects: Skidmore, Owings & Merrill, Chicago. Engineer: L. Boduroff, Denver. Prestressing: Prescon Corp., Corpus Christi, Texas.

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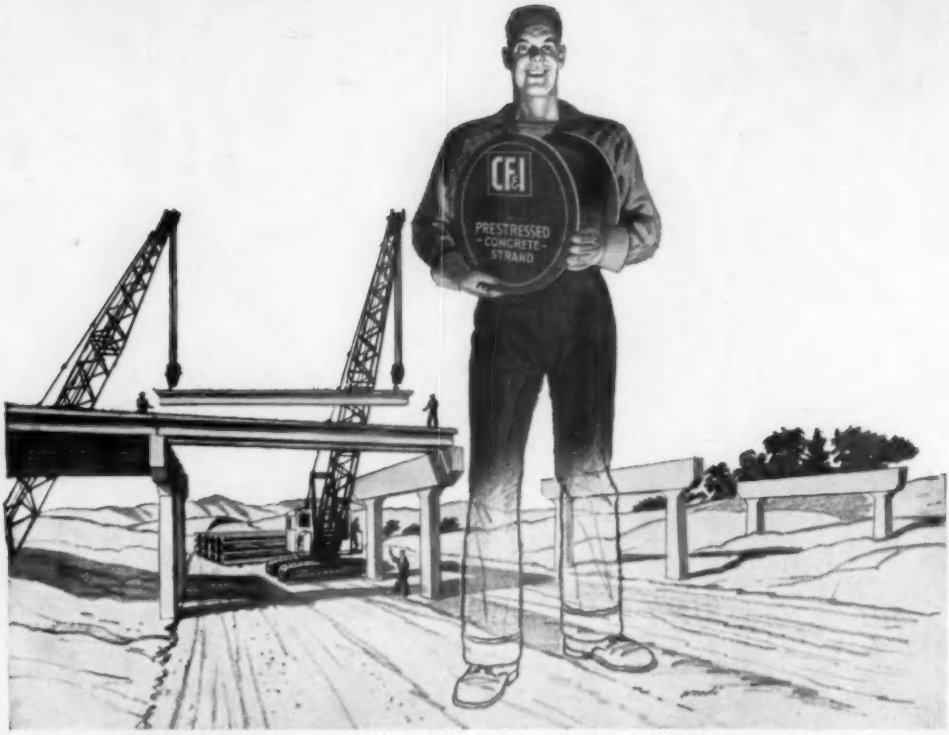

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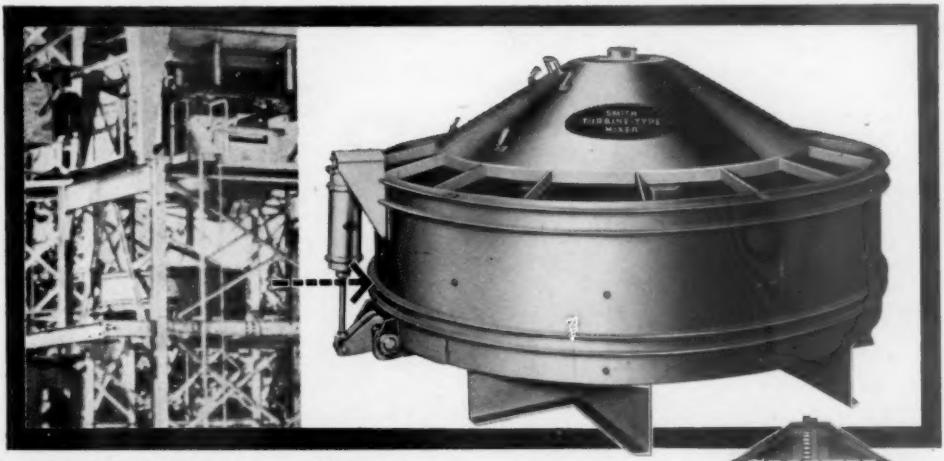

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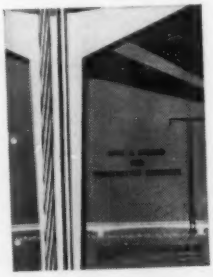

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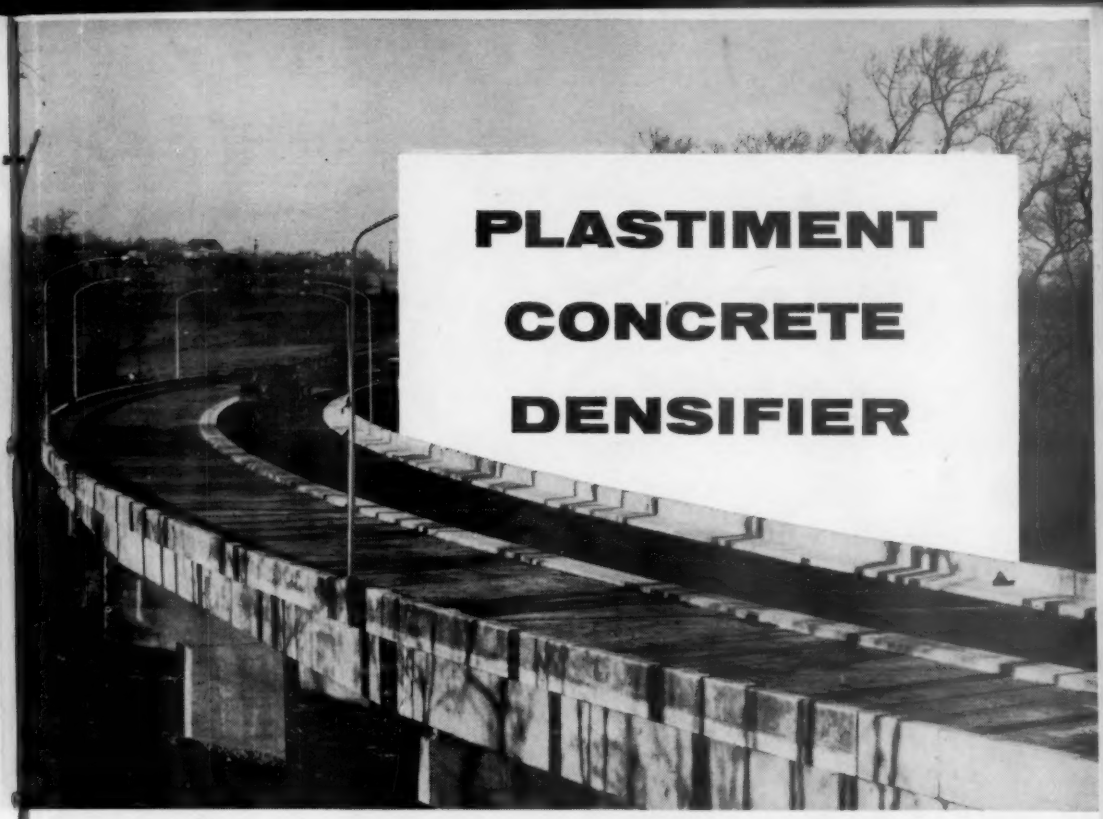

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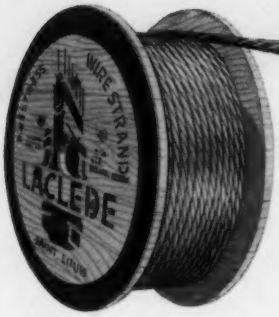

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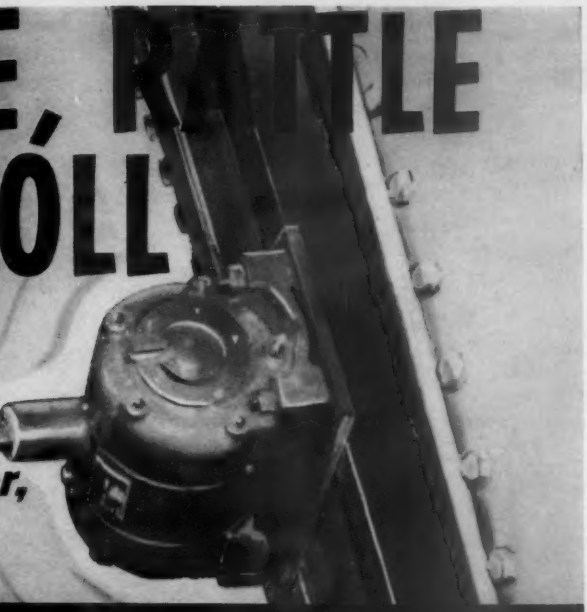

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LESCHEN PRESTRESS STRAND

The New England Concrete Pipe Corporation has built 200 bridges of prestressed concrete in the past six years. One of their most recent jobs pictured here under construction is the Broadway Bridge, one of nine bridges on the East Providence Expressway, between Providence and East Providence, R. I. The Broadway Bridge required eighteen girders, 45″ in depth, measuring 67′2″ in length and weighing 20 tons each. Owner: State of Rhode Island. Design engineer: Chas. A. Maguire & Associates, Providence, R. I.

New England's largest concrete prefabricator, New England Concrete Pipe Corp. uses wire strand at a rate of 20,000 feet daily; and can be truly considered experts on wire strand quality. Quoting their Vice President, Mr. Bierweiler, "It is obvious that a public project, subject to inspection by the State of Rhode Island and usage by literally millions of citizens, must meet the most exacting and rigid tests, and must be constructed in accordance with the most advanced

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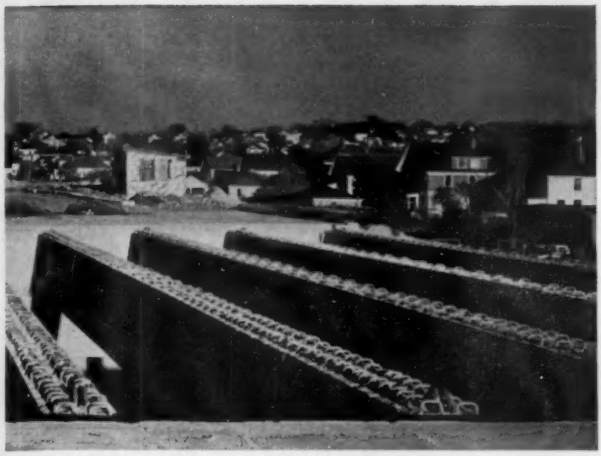

Bridge job showing New England Concrete Pipe Corp. concrete girders constructed with ⅜" Leschen Prestress Strand. Each seven-wire strand has a tension of 7 tons for a total 340-ton squeeze of prestressed force in each girder.

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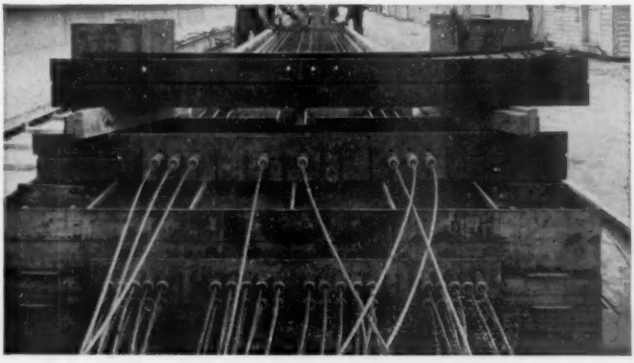

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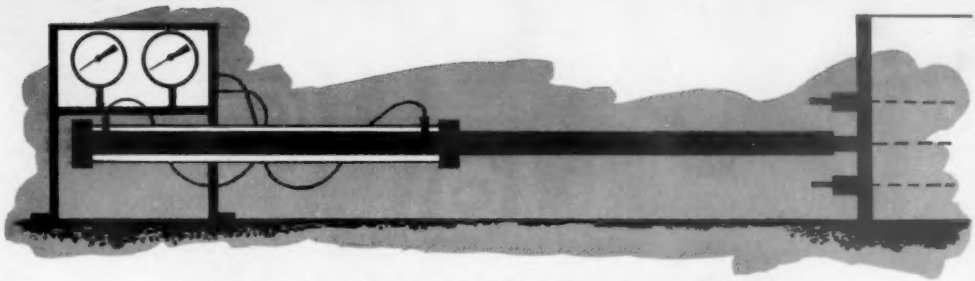

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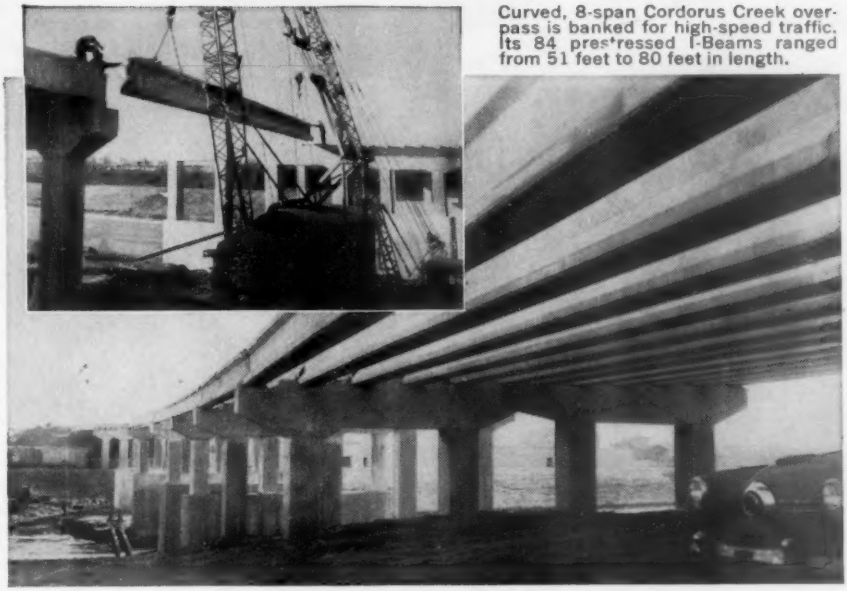
1,000 Prestressed Bridges!



Curved, 8-span Cordorus Creek overpass is banked for high-speed traffic. Its 84 prestressed I-Beams ranged from 51 feet to 80 feet in length.

● Starting ten years ago with the first prestressed concrete bridge in America —the famed Walnut Lane Bridge in Philadelphia—Pennsylvania now has over 1,000 prestressed bridges in place. Under a recent policy of alternate bidding, 75 out of 80 contracts for spans up to 100 feet in length have been awarded to prestressed concrete.

The newest of these projects is a series of five bridges on the York By-Pass of the new Harrisburg-Baltimore expressway. These bridges required 188 prestressed I-Beams—the first pretensioned, deflected-strand I-beams used in the Northeast. All beams were manufactured by Pennsylvania Prestress, Inc., York, Pa., with 6000 psi concrete, using 'Incor' High Early Strength Cement.

With modern highway structures like this, the Pennsylvania Highway Department gains not only the traditional advantage of concrete's low maintenance cost, but also *low initial cost.*

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EDITORIAL PAGE

JOURNAL

OF PRESTRESSED CONCRETE INSTITUTE

Vol. 4 September, 1959 No. 2

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While the greatest use of prestressed concrete is, and will continue to be, in the construction of buildings, prestressed concrete is filling a large and growing place in highway bridge construction. Much of the pioneering was done in Europe, but it has been in this country that our peculiar talent for organization and mass production has made a manufacturing process of it. The manufacture of structural members, in this sense, requires costly plants and equipment. These plants can only operate economically if they have volume production and this can come only through standardization. To meet this obvious need the Prestressed Concrete Institute and the Bridge Committee of the American Association of State Highway Officials joined forces and through joint committees have designed and distributed recommended standard plans for I-beams, box girders, slabs and channel sections. The number of different sections is kept to a reasonable minimum to keep plant costs down. The economy of repeated use of forms and equipment overbalances any minor gains of refined design. The use of these standards in highway bridge design is recommended.

A vote of thanks is due to Mr. William E. Dean, Assistant State Highway Engineer of Florida, Chairman of the committee, and to the men of the P. C. I. and AASHO who worked with him.

G. S. Paxson, Chairman
Committee on Bridges & Structures, American Association of State Highway Officials.

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TEST OF A PRESTRESSED CONCRETE DOUBLE-TEE SLAB

at Ecole Polytechnique, Montreal

Designed by:
Claude Lanthier, P. E.

Sponsored by
Pressure Pipe Co., of Canada, Ltd.

Test Conducted by
Professor J. E. Hurtubise, P. E.
Professor B. A. Heskith, P. E.

This slab was received at the Structural Laboratory of Ecole Polytechnique on January 19, 1959.

Description of slab—The slab, consisting of an upper flange and two webs, was described as being prestressed by means of twelve strands, each consisting of seven high tensile strength steel wires (ASTM Designation: A-416-57 T). Details and dimensions were given as follows:

weight of slab: 47 lb. per sq. ft.
length: 45 ft. 8 in.
width: 3 ft. 11⅞ in. (4 ft. nominal)
thickness of flange: 2 in.
number of webs: 2
depth of webs: 12 in.
width of webs: 2½ in. at bottom to 4½ in. at top
spacing of webs: 2 ft. c/c
number of strands per web: 6
nominal diameter of strands: ⅜ in.
area of steel per strand: 0.0799 sq. in.

Five of the strands were described as being deflected at the third points of the slab, the distance from the lower edge of the web to the centre-line of the strands being as follows:

	end section	center section
1st strand	1½ in.	1½ in.
2nd strand	3 in.	2½ in.
3rd strand	4½ in.	3½ in.
4th strand	7½ in.	4½ in.
5th strand	9 in.	4⅞ in.
6th strand	10½ in.	5¼ in.

The slab was made of concrete having a minimum ultimate strength of 6,000 psi at the age of 28 days. The age at time of the test was given as 30 days.

The slab was designed in accordance with the "By-Law concerning the Erection of Buildings in the City of Montreal" (#1900) and details given as follows:

Allowable compressive stress in concrete 0.40f'c

Allowable tensile stress in concrete 0.03 f'c

Working tensile stress in steel 150,000 psi

Design superimposed load 50 lb. per sq. ft. (Snow: 40 lb. per sq. ft.—roofing, insulation, etc.: 10 lb. per sq. ft.).

Total load, including own weight—97 lb. per sq. ft.

Testing procedure—For testing, the slab was placed on fixed steel cylindrical supports with a free span of 45 ft. Steel plates, 2½ in. wide, had been embedded into the webs during manufacturing, these plates providing steel-on-steel supports.

The loading was done by means of Haydite concrete blocks measuring 12 in. x 16 in. x 8 in. and weighing 34 lbs. each.

These blocks were placed on the slab in transversal rows, leaving a gap of approximately 1½ in. between consecutive rows in order to avoid an arching effect.

Measurements of deflection at mid-span on both sides of the slab were taken by means of a high precision surveyor's level giving readings to the nearest 1/64 in.

The loading procedure was carried out in accordance with the Tentative Recommendations for Prestressed Concrete. (ACI-ASCE Joint Committee 323. Title No. 54-30). These recommendations suggest, as a minimum, the greatest of the two loadings:

1.2 D + 2.4 L, corresponding to a superimposed load of 129 lb. per sq. ft.

or 1.8 (D + L), corresponding to a superimposed load of 128 lb. per sq. ft.

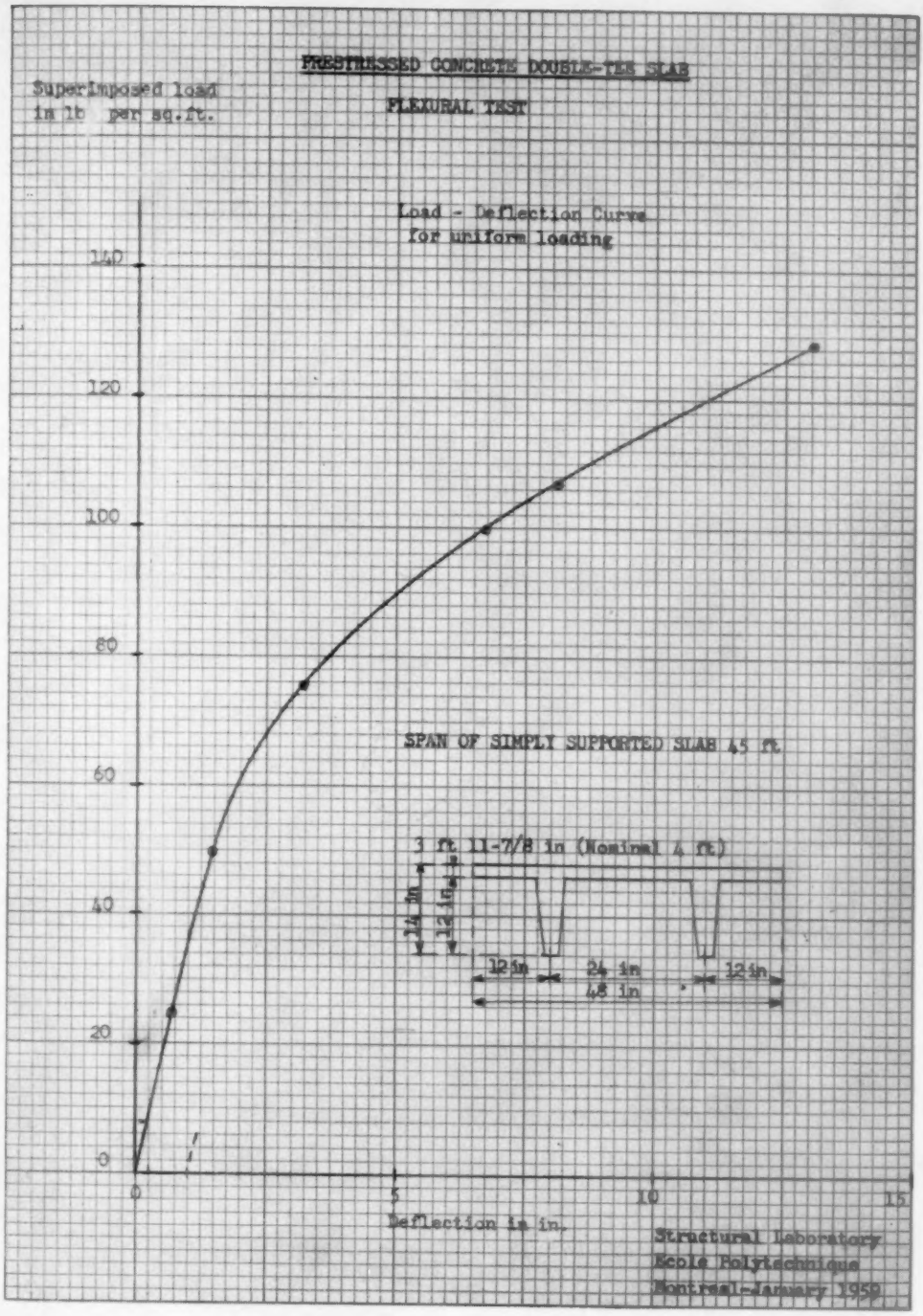
Test results.

Elapsed time	Superimposed load lb. per sq ft	Deflection in inches Average of 2 sides of slab	Comments
0	0	0	Camber 2-3/64 in.
	24.2	22/32	
15 min.	50	1-37/64	
30 min.	0	1/32	Residual camber 2-1/64 in.
0	0	0	Reset to zero
	25	21/32	
	50	1-15/32	
	75.6	3-13/64	First hair cracks
	100	6-47/64	Additional hair cracks
1 h 15 min.	107	8-7/64	Numerous hair cracks
	129	12/63/64	No cracks greater than 1/100 in.
1 h 45 min.	0		All cracks closed. Time reset to zero
15 min.		1	
1 h		31/32	
3 h 45 min.	0	15/16	
17 h 15 min.	0	7/8	Remaining camber 1-9/64 in.

Comments: It is noted that the slab has withstood without failure the loading recommended by the ACI-ASCE Committee.

The deflection under the design load was 1-15/32 in. or 1/368 of the span, while the deflection under the maximum load was 12-63/64 in.—The residual deflection, 17 hr. 15 min. after unloading, was found to be 7/8 in. or 1/617 of the span.

The calculated deflection under a live load of 40 lb. per sq. ft. would be 1-11/64 in. or 1/461 of the span.


PRESTRESSED CONCRETE DOUBLE-TEE SLAB
FLEXURAL TEST
Load - Deflection Curve
for uniform loading
Superimposed load
in lb per sq.ft.
140
120
100
80
60
40
20
0
SPAN OF SIMPLY SUPPORTED SLAB 45 ft.
3 ft 11-7/8 in (Nominal 4 ft)
1/4 in
1/2 in
12 in
24 in
48 in
0
5
10
15
Deflection in in.
Structural Laboratory
Ecole Polytechnique
Montreal-January 1959

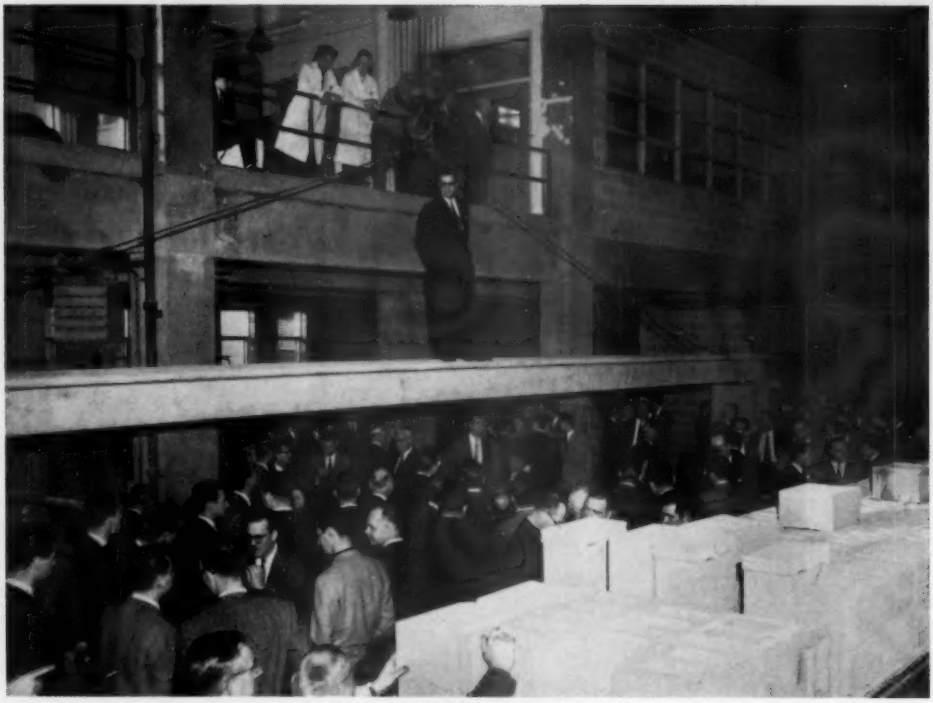


Fig. 1: Explanation of test given.

Fig. 2: Loaded to 1.8 D.L., 2.4 L.L.

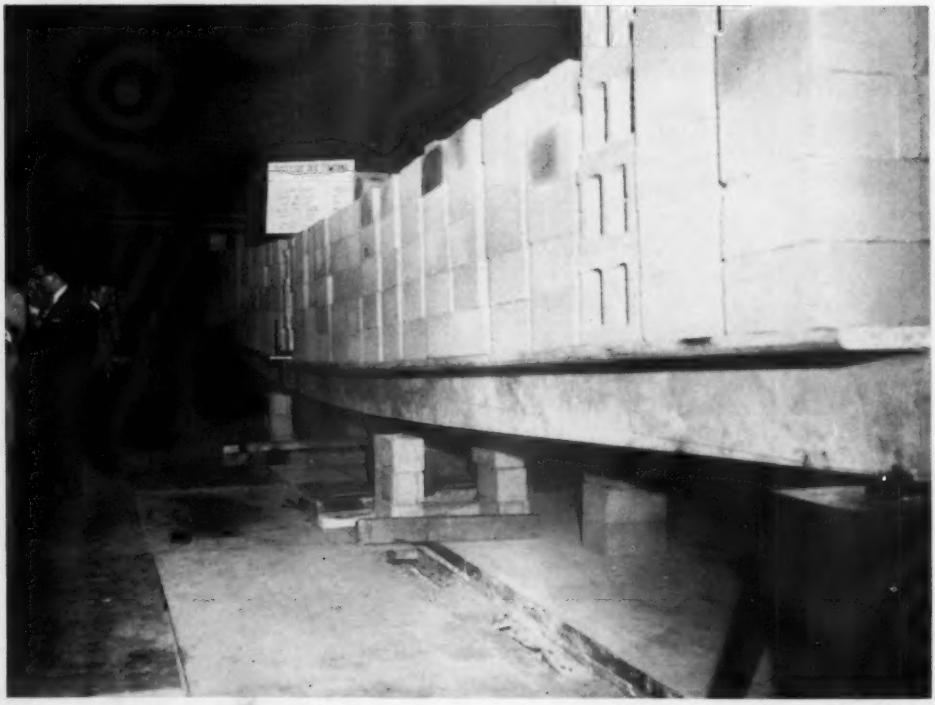
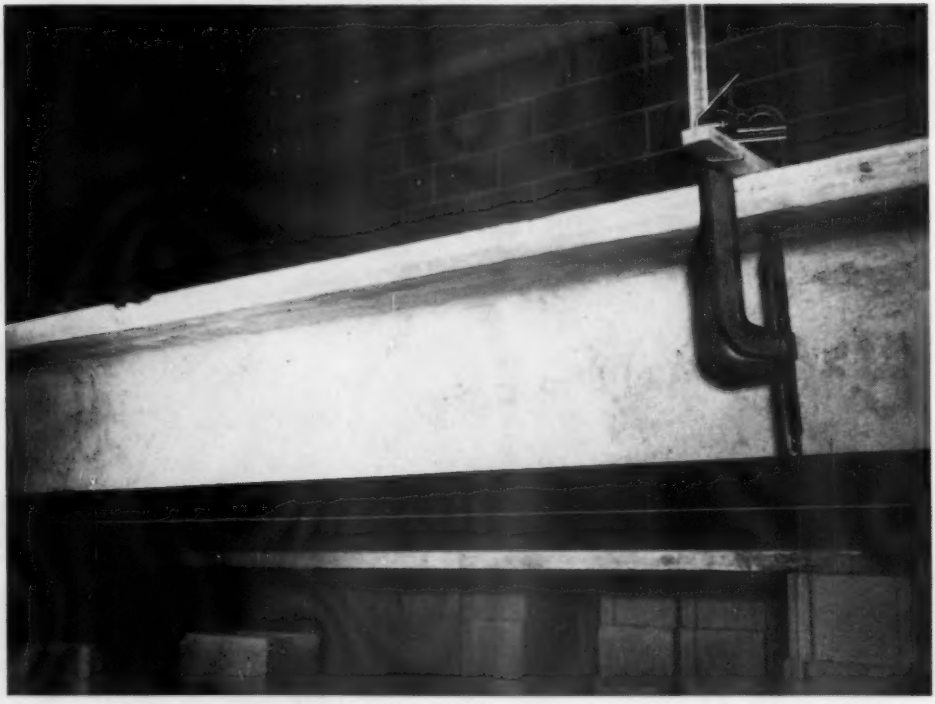


Fig. 3: Camber recovery after being unloaded from 1.2 D.L., 2.4 L.L.

Fig. 4: Member loaded to 2 times Montreal Code Loading

Fig. 5: Tensile cracks at 1.2 D.L., 2.4 L.L.

Fig. 6: Cracks completely closed after unloading



THE APPARENT MODULUS OF ELASTICITY OF PRESTRESSED CONCRETE BEAMS UNDER DIFFERENT STRESS LEVELS

W. N. Lofroos (1)

A. M. Ozell (2)

INTRODUCTION

The application of a sufficiently large prestressing force at a point below the neutral axis of a simply supported prestressed concrete beam will cause an upward deflection of the beam. This upward deflection is defined as camber. If the beam remains under these conditions and no additional loads are applied there will be some losses in the prestressing force due to shrinkage of the concrete and other factors. Normally under these conditions a decrease in camber would be expected, but in a prestressed concrete beam the opposite occurs. The camber of the beam will continue to increase as a function of time. This phenomenon is believed to be principally the effect of creep in the concrete.

Creep can be defined as the total time-dependent change in strain minus the shrinkage. A number of studies have been made to determine the rate and magnitude of creep in concrete. It has been shown that plain and reinforced concrete cylinders loaded in compression have continued to creep throughout a thirty year test period. (1)* It has also been demonstrated that the creep in cylinders at low stress levels is directly proportional to the stress in the concrete. (2) This linearity does not exist in cylinders loaded to a stress which is greater than 20-30% of their ultimate strength. Stresses which exceed this amount are commonly found in prestressed concrete.

Tests have been conducted to determine the effect of creep on the loss of prestressing force. (3) These losses were determined experimentally as creep coefficients or percentages of the initial elastic strain. The use of these factors as a method of calculating stress loss in the prestressing bars or strands has been generally accepted in design practice. (4)

Consider now the effect of creep on the Modulus of Elasticity of the concrete. The load-deflection relationship for a prestressed concrete beam would be linear up to the cracking load. This indicates that the Modulus of Elasticity of the beam is constant within this range. If the load-deflection relationship is studied over a period of time a different set of conditions will exist. Here the camber is increasing with respect to time, and the prestressing force is decreasing. From these two factors alone it can be seen that the Apparent Modulus of Elasticity of this beam is not a constant, but a function of time.

All of the factors that have been discussed up to this point enter into the problem of computing the deflections of prestressed concrete beams. There is no known way of determining the Modulus of Elasticity of the beam to any degree of accuracy. Cylinder tests will give an indication of the true value only. The actual beam Modulus is dependent on age, stress level, curing conditions, mix and many other factors. Also any deflection which has to be computed over a period of time becomes a function of the concrete creep. This creep is a function of stress level, moisture conditions, age and other factors. All this points out the extreme interdependence of a large number of variables in this one behavior alone.

Some recommendations have been made suggesting values of the Modulus of Elasticity to be used under various conditions. Where possible, these values will be used as a comparison with the results obtained in this study.

OBJECTIVE AND SCOPE OF THE STUDY

The primary purpose of this study is to present, from the results of a series of tests, the following factors:

1. The relationship between the beam Ap-

(1) Graduate Student, University of Florida, Gainesville.
(2) Professor, Civil Engineering Dept., University of Florida

*Numbers refer to references.

parent Modulus of Elasticity, the cylinder Modulus of Elasticity, and the beam Modulus of Elasticity;
2. The relationship between the total strain in the concrete, the creep, the shrinkage, and time;
3. The relationship between creep and the Apparent Modulus of Elasticity;
4. The effect of two different stress levels on all the above.

To accomplish this four beams were tested. During the primary testing period, the beams were loaded with only the prestressing force and their own dead load. The only condition varied was the prestressing force. Beams 1 and 2 were tested at approximately 0.40 f'_{ci}. Beams 3 and 4 were loaded to about 0.25 f'_{ci}. All other factors were kept as nearly constant as possible.

For the purpose of this series of tests the Apparent Modulus of Elasticity will be the Modulus as computed from the magnitude of camber and post-tensioning forces.

DESCRIPTION OF THE SPECIMENS

Four identical beams, 10 in. x 12 in. x 25 ft.-0 in. center to center of supports were cast for this test. The beams had a net area of 113.8 in.^{2} and a net moment of inertia of 1396 in.^{4}. Rollers were provided at the beam ends so that it was possible to cast the beams, instrument, and complete all primary testing without moving the beams. No stirrups were used. Pick-up hooks were provided at both ends of each beam.

The following design mix was used for the specimens:

Cement	620 lbs.
Coarse Aggregate	1798 lbs.
Fine Aggregate	1124 lbs.
Water	38.32 gals.

The fine aggregate was Interlachen sand with a fineness modulus of 2.14 and a specific gravity of 2.63. The coarse aggregate was Brooksville stone, one inch maximum size, with a specific gravity of 2.53. The concrete was provided by a local ready mix plant.

Internal vibration was used during the casting of the beams to facilitate the placing of the concrete. All of the specimens were cured for five days under wet burlap. At the end of this curing period the cylinder strength was in excess of 4800 psi. As this strength was satisfactory for the tests, no further curing was required. The forms were therefore removed and instrumentation begun at this time.

Prior to the pouring of the concrete it was necessary to place three Duoflex casings longitudinally in each of the beam forms. These casings were used to form the holes through which the post-tensioning bars were run. The holes were located so that the center of gravity of the post-tensioning steel was at the lower kern point of the beam.

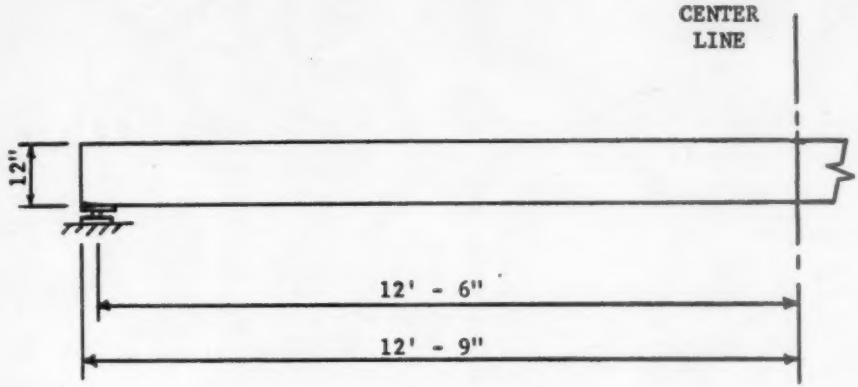
Three straight, unbonded, Stressteel bars, ¾ in. in diameter were used to provide the prestressing in each of the beams. These bars had a Modulus of Elasticity of 28.2 x 10^6 psi and a yield strength of 0.2% offset in excess of 130,000 psi.

Two shrinkage specimens, 10 in. x 12 in. x 5 ft.-0 in., were cast at the same time as the beams. They were cured and stored, at all times, under the same conditions as the beams. As these specimens were unstressed there would be no creep effect and any change in their strain would correspond to the change in strain of the beams due to shrinkage and temperature.

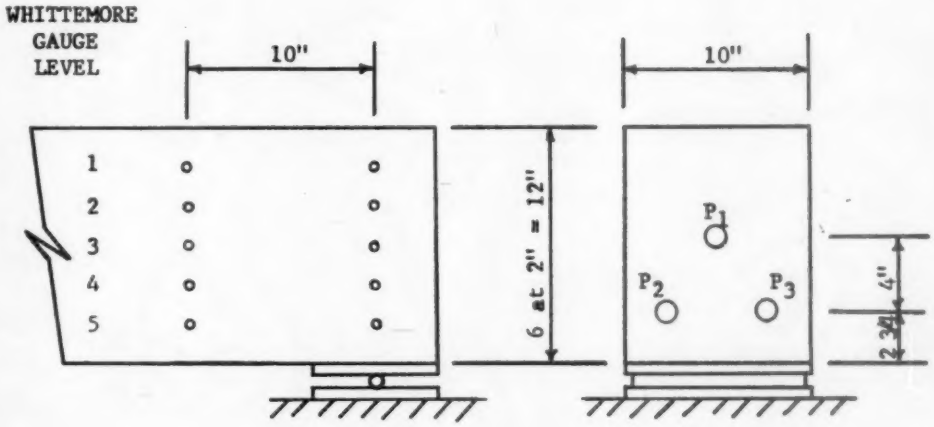
Thirty standard 6 in. by 12 in. cylinders were cast at the time of the concrete pour. All of these cylinders were cured for five days under wet burlap. At this time the forms were stripped and a few of the cylinders were placed in a curing room with a constant temperature of 70°F and a humidity of 100%. These cylinders were used to obtain the concrete properties at these ideal conditions. The remainder of the cylinders were stored under the same conditions as the beams and the shrinkage specimens, and were used to obtain the properties of the concrete under test conditions.

INSTRUMENTATION OF THE SPECIMENS

For convenience the beams were numbered 1 through 4. Beams 1 and 2 were tested as a pair at the higher stress level while beams 3 and 4 were tested as a second pair at the lower stress level. As the post-tensioning bars were jacked from one end only, that end was called the near end. The other end was called the far end. The post-tensioning bars and the Whittemore gauges were numbered as shown in Figure 1.

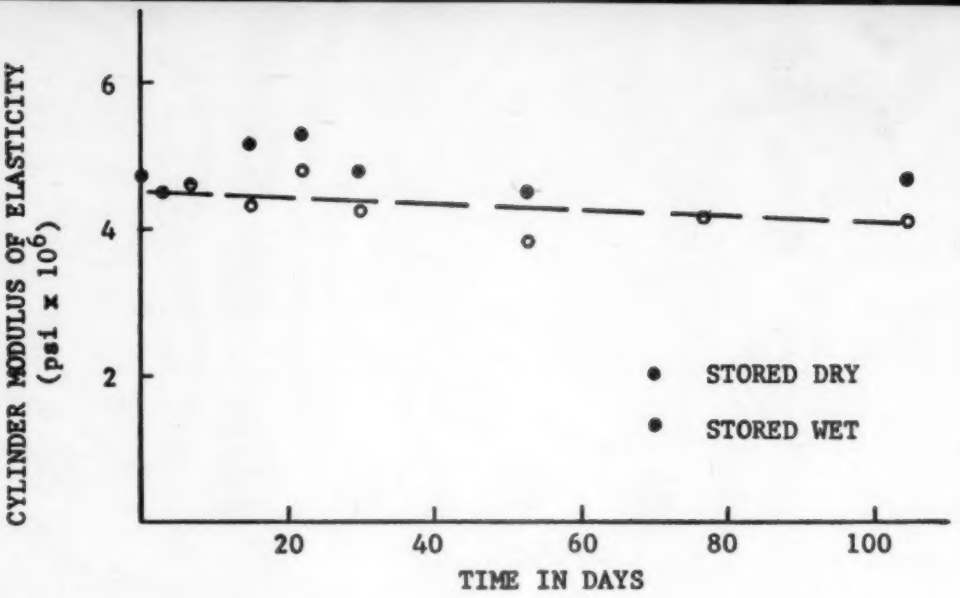


HALF SIDE VIEW



TYPICAL END VIEWS

FIG. 1 - BEAM DETAILS



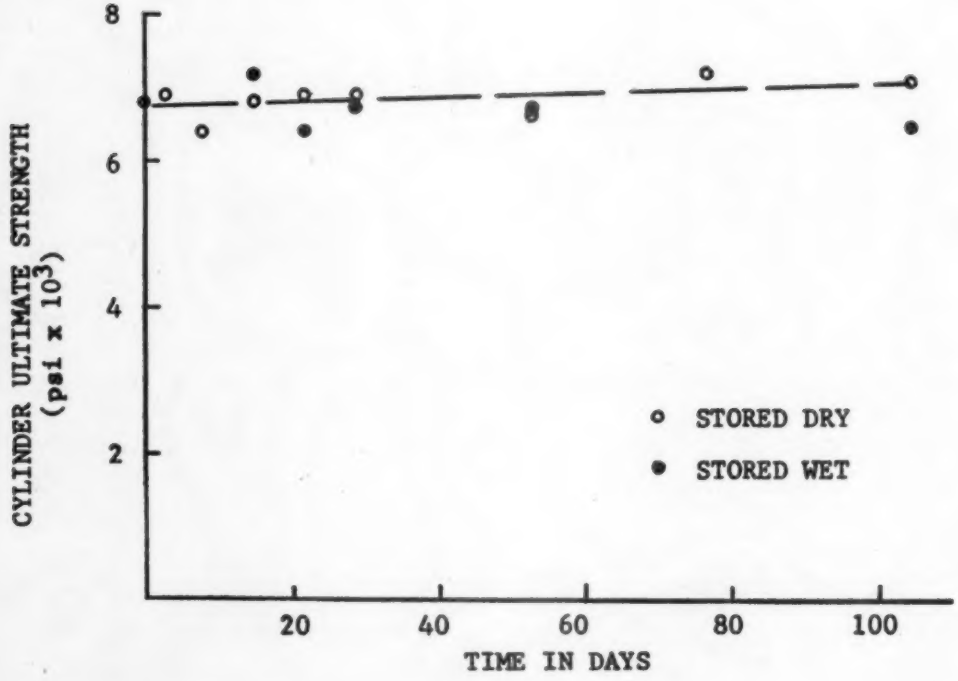
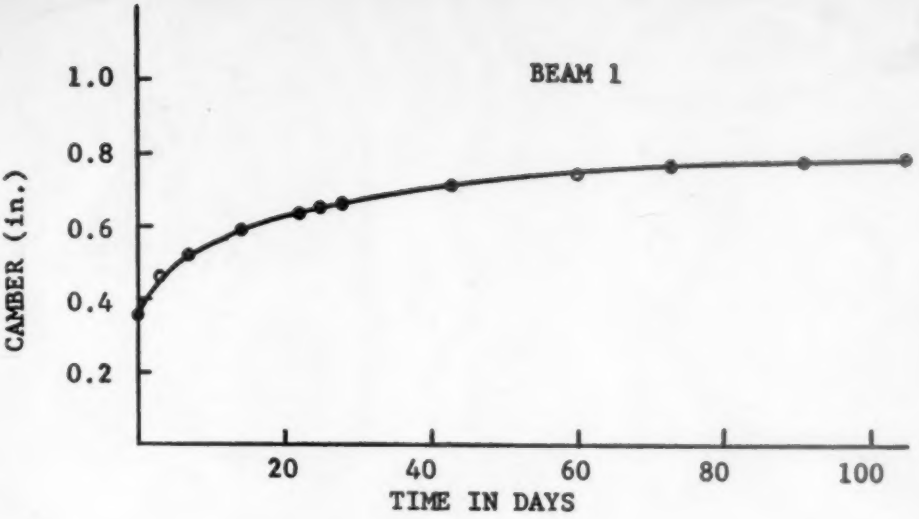


FIG. 2 - CONCRETE PROPERTIES



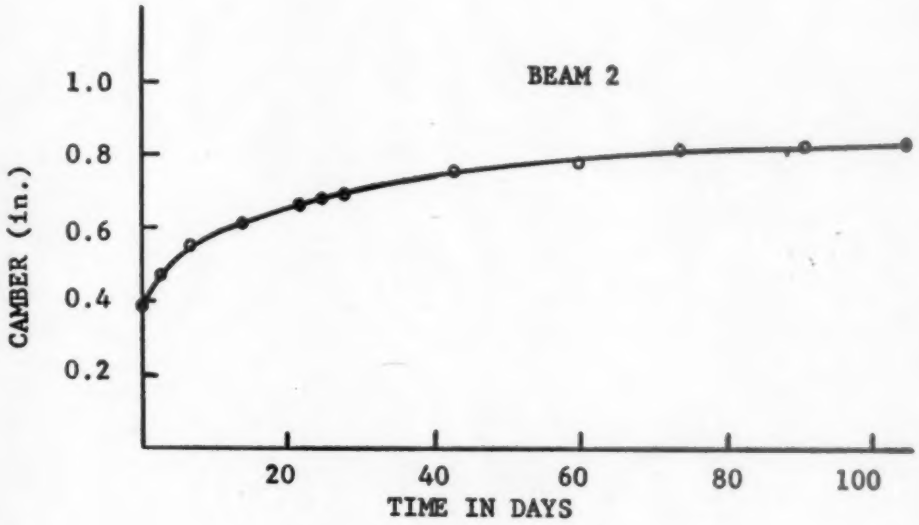
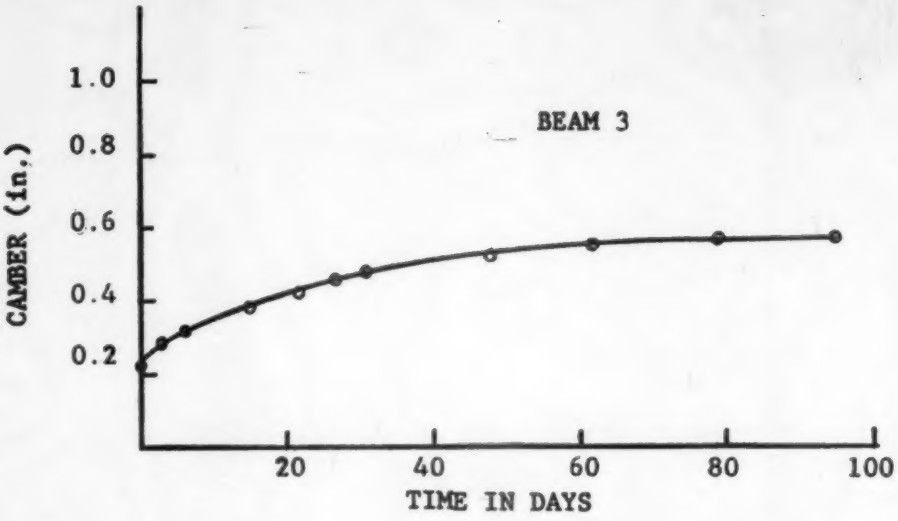


FIG. 3 - CAMBER CURVES, BEAMS 1 AND 2



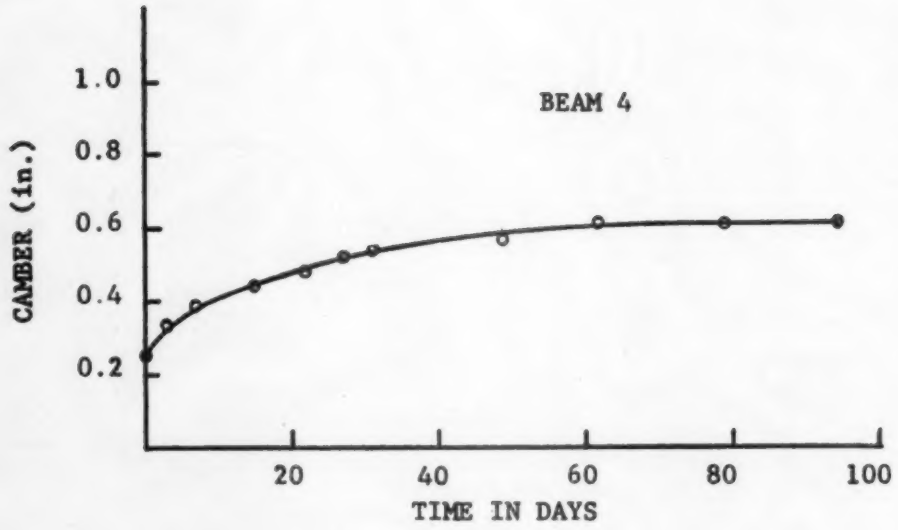


FIG. 4 - CAMBER CURVES, BEAMS 3 AND 4

Baldwin Lima Hamilton SR 4 Strain Gauges (type PA 3) were attached to each end of the post-tensioning bars. After the bars were in place these gauges were located about 12 in. inside the beams. The gauges were thoroughly waxed and covered with a wrapping of sheet rubber to protect them from abrasion and moisture. As the Stressteel system for post-tensioning requires that the ends of the bars be anchored in bearing plates, small grooves were cast in the ends of the beams leading from the sides to the Duoflex casings to provide access holes for the lead wires to the electrical strain gauges.

The strains in the bars were read directly from these gauges with a Baldwin Lima Hamilton SR 4 strain indicator. This instrument gave the strain in the bars directly in microinches. As the Modulus of Elasticity of the steel was known the load in each bar could be accurately determined. All electrical gauges were corrected for zero drift.

Ames dials, reading to 0.001 in., were used to measure the deflection at midspan of the beams. The dials were attached to brackets which were bolted to the beams. Steel plates were grouted to the floor for the dials to bear on. With this arrangement the vertical deflection was measured directly.

All concrete strain measurements were made with a Whittemore gauge. A 10 in. gauge length was used throughout the test. The gauge points were located at the ends and at midspan of both sides of each beam. In order to obtain a vertical strain distribution the gauges were placed at different levels as shown in Figure 1. The gauge points were placed at levels 1, 3 and 5 at all six locations on each beam. Levels 2 and 4 were instrumented at the near end and the centerline of the east side of each beam only. These gauges served as a check on the strain distribution and also provided additional concrete strain data. The shrinkage specimens were instrumented with Whittemore gauge points at all five levels at their midspan on both sides.

TEST PROCEDURE

The four beams were tested under the effect of the post-tensioning forces and their own dead load only. Periodic readings were taken to determine the change in the camber, post-tensioning forces, and the concrete strain. The specimens were tested in place and were not moved until the load-deflection tests were conducted at the end of the testing period.

The actual test period for a beam began at the time it was post-tensioned. Readings were taken on all gauges just prior to the pulling of the bars. The beam was then prestressed and the first set of data was taken. For recording purposes this was the zero day reading for that beam.

A definite creep effect was expected. As a typical creep curve shows a rapid rate of change during its early period, subsequent readings were taken at 3, 7, 14, 28, 60, and 90 days. In some cases readings were taken at more frequent intervals.

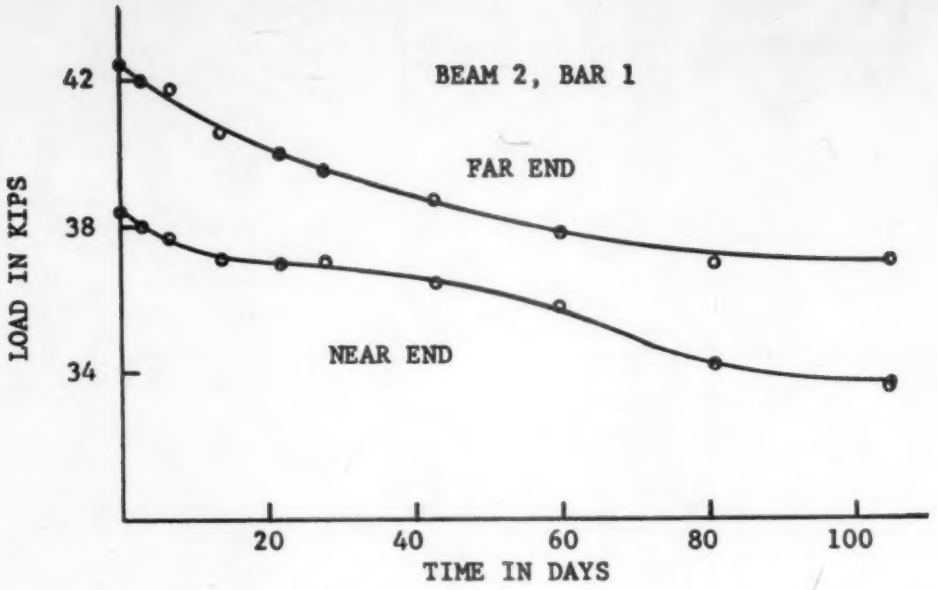
To eliminate, as much as possible, the effect of local temperature changes, all readings were taken during that time of the day when the ambient temperature in the laboratory was 72°F.

The beams were post-tensioned with a Stressteel Hydraulic Center Hole Tensioning Unit. The load on each bar was checked by three methods; measuring the total elongation of the bar, measuring the unit elongation of each bar with the electrical strain gauges, and reading directly from a calibrated indicator on the jack. All bars were overstressed a small amount then released to the proper load. All initial elongation readings were made with a load of 500 lbs. on the bars to insure that the anchor end was seated properly.

Beams 1 and 2 were post-tensioned 20 days after they were poured. The computed bottom fiber stress at the ends of the beams at this time was 2400 psi. Beams 3 and 4 were post-tensioned 12 days later. Their computed fiber stress was 1500 psi. For both pairs of beams, the method of test was identical. The only difference between the two pairs was the prestressing force applied.

TEST RESULTS

The properties of the concrete used in this test are given in Figure 2. During the entire testing period both the ultimate strength and the Modulus of Elasticity of the cylinders remained almost constant. Stress-strain curves were plotted for the cylinders. These curves were straight lines up to stress values well above those in the beams. The storage conditions apparently had little effect on the ultimate strength, but the cylinders stored with the beams had a slightly lower Modulus of Elasticity than



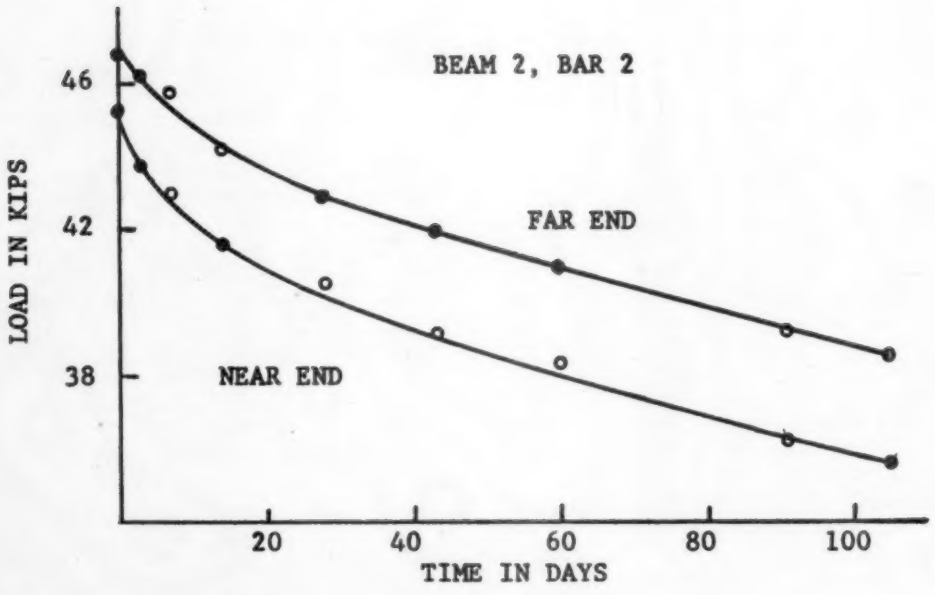
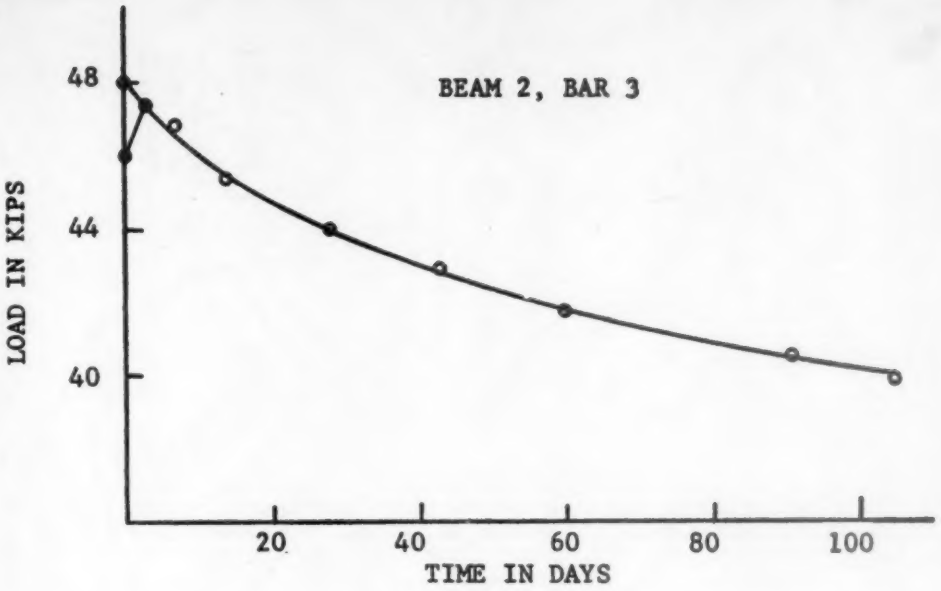


FIG. 5 - POST-TENSIONING FORCES, BEAM 2



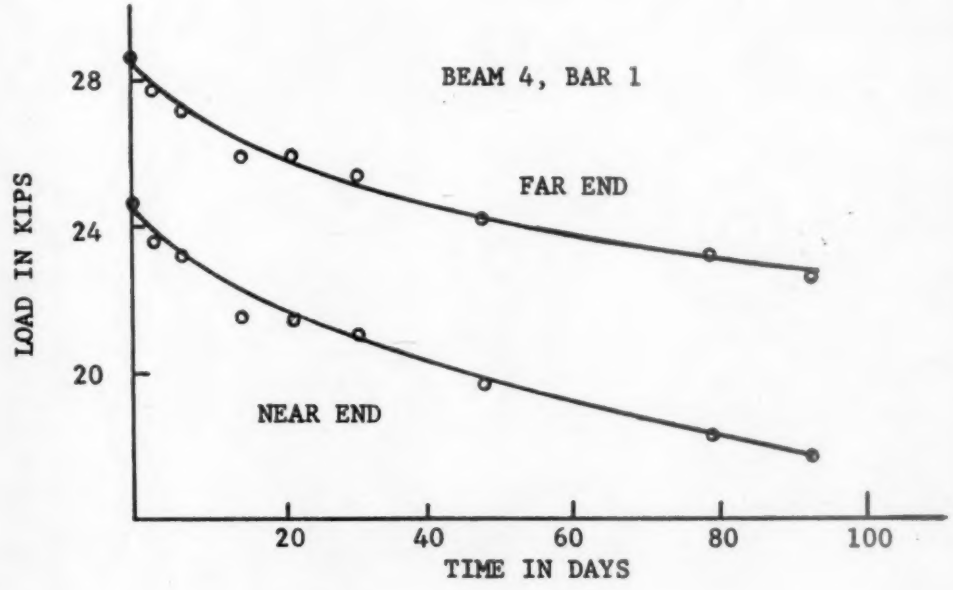
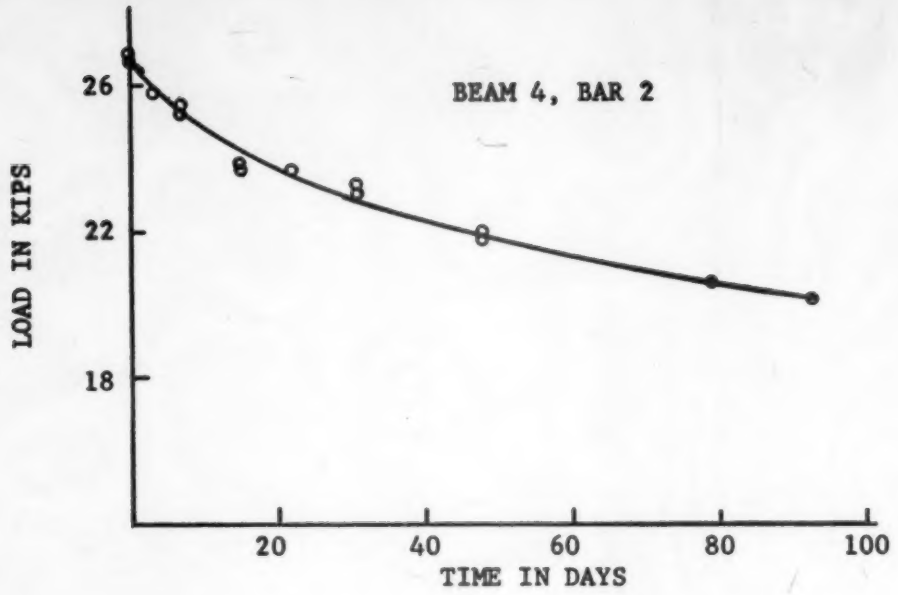


FIG. 6 - POST-TENSIONING FORCES, BEAMS 2 AND 4



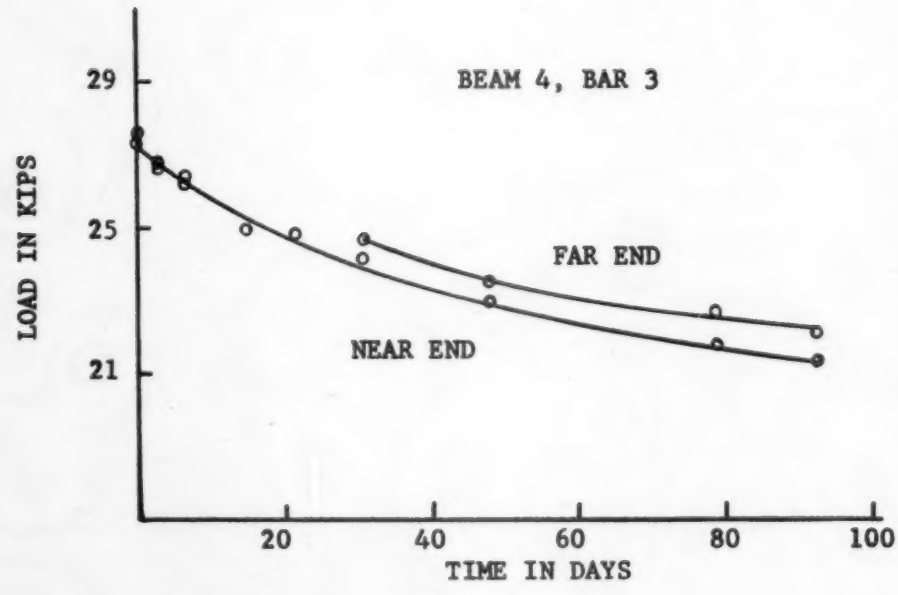
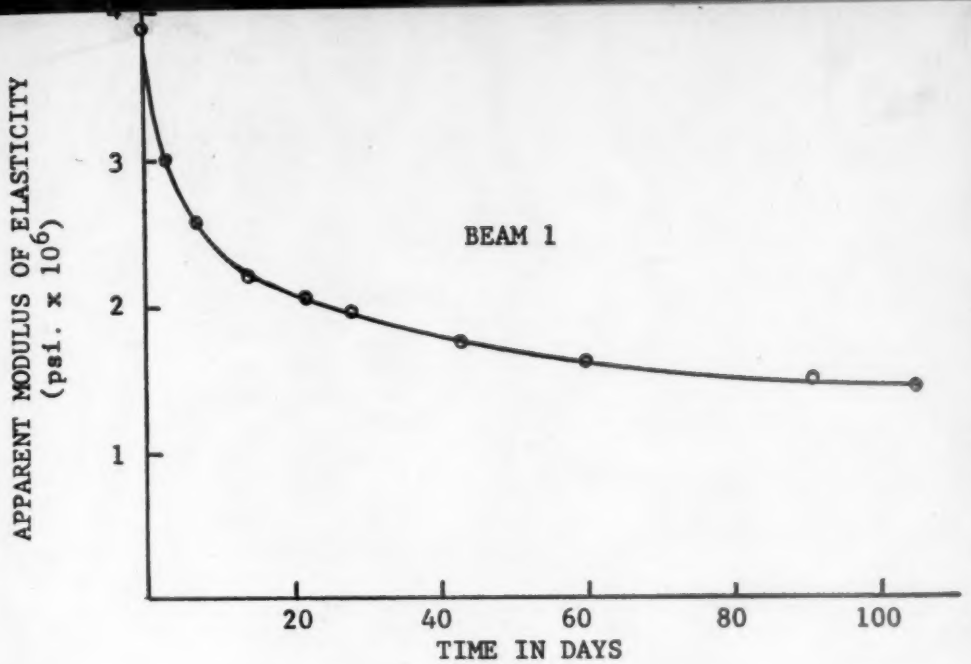


FIG. 7 - POST-TENSIONING FORCES, BEAM 4



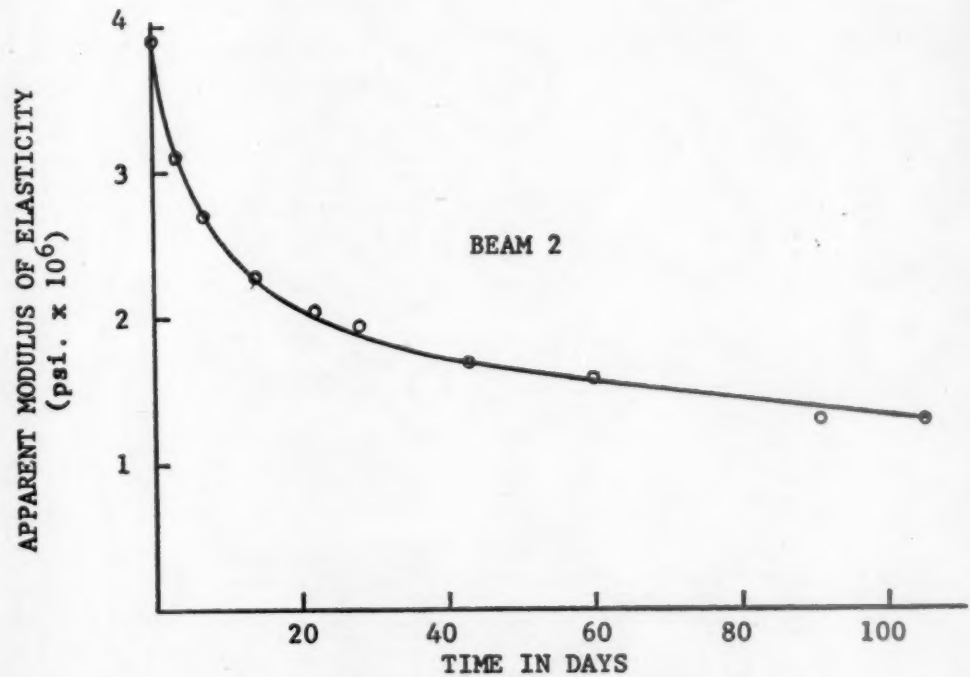
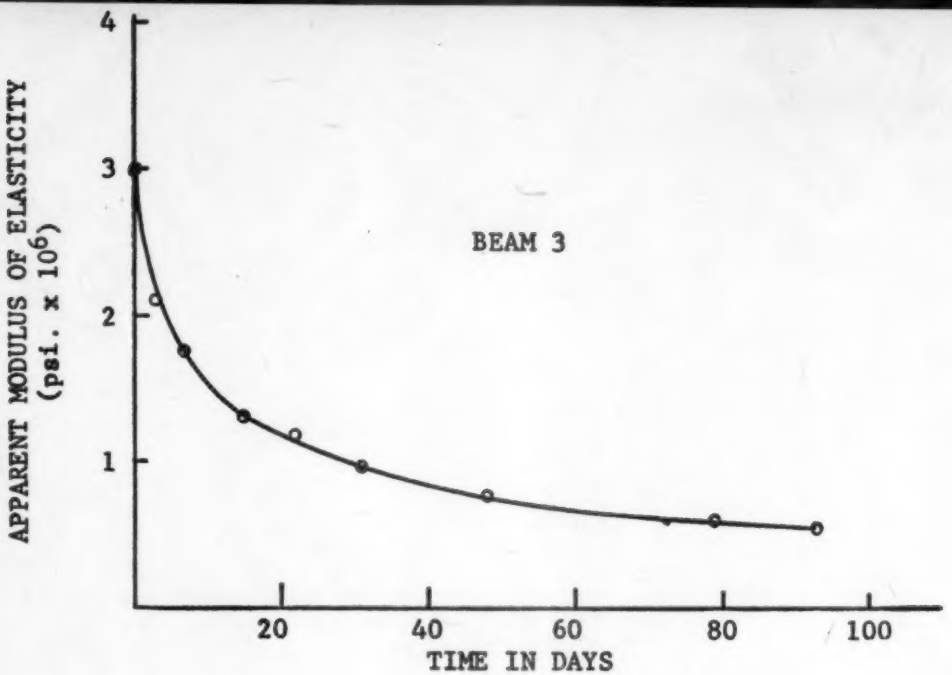


FIG. 8 - APPARENT MODULUS OF ELASTICITY, BEAMS 1 AND 2



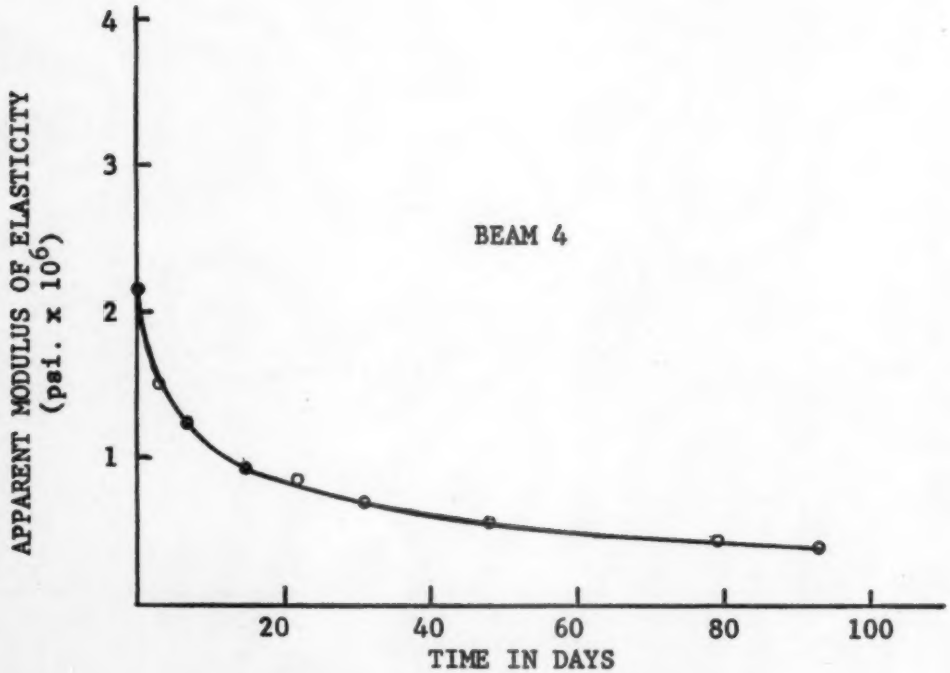


FIG. 9 - APPARENT MODULUS OF ELASTICITY, BEAMS 3 AND 4

those stored in the curing room. The cylinders that had been stored in the curing room were tested immediately after their removal from this storage area.

The camber curves for the four beams are given in Figures 3 and 4. The camber showed a large rate of increase during the early portion of the testing period, but by the end of the test it was increasing very slowly. The percentage increase in camber is of particular interest as it has a definite effect on the Apparent Modulus of Elasticity curves. Beams 1 and 2 had an initial mean camber of 0.375 in. and a camber at 90 days of 0.798 in. This was an increase of 108% of the original value. Beams 3 and 4 had a mean initial camber of 0.238 in and a final camber of 0.584 in. for an increase of 146%. Therefore, even though beams 1 and 2 gave the higher initial and final values, their percentage of increase was less than that of the beams stressed at the lower stress level.

Typical load versus time curves for a number of the post-tensioning bars are given in Figures 5 through 7. The bars for beam 2 all showed a definite friction loss at the time of prestressing. This loss was expected as the conduit allowed only ⅜ in. clearance around the bars. As the initial camber exceeded this value, it was expected that the bars would be in contact with the beam at the center. Bar 3 was apaprently able to overcome the frictional resistance sometime during the first three days by slipping, because for the remainder of the test the values of stress at both ends were identical. The initial camber for beams 3 and 4 did not exceed the ⅜ in. clearance around the bar, so no friction loss was expected. For some reason this was not true for bar 1 which showed a definite loss throughout the test. Bars 2 and 3 gave values from both ends which were very close, but after the camber had increased so that the bars were touching the conduit there was some variance. This was particularly noticeable in bar 3 after the 31-day reading.

As in the camber curves, there is a point of interest here that should be noted, that is, the percentage of stress loss in the beams. The mean prestressing loss for beams 1 and 2 at 90 days was 14.2%. Beams 3 and 4 had a loss of 23.9% over the same period. This shows again that the beams with the higher initial prestressing had a lesser percentage change than did the beams at the lower stress level.

The Apparent Modulus of Elasticity is computed by using the Moment-area method. The moment diagrams of the beams, the section properties, and the deflections are all known. The only unknown is the Modulus of Elasticity. As these values are to be used primarily for comparison, a number of simplifying assumptions were made. The eccentricities of the post-tensioning bars were considered constant and the effect of the bending upward of the bars was neglected. The equation derived using this method is:

$$E_a = 27.04\,(P_2 + P_3)/c - 5.21\,P_1/c - 0.788 \times 10^6/c$$

E_a = Apparent Modulus of Elasticity (psi)

P_n = Total stress in bar "n" (lbs.)

c = Camber (in.)

The curves determined by this equation are given in Figures 8 and 9.

It is apparent that this equation does not represent the true elastic properties of the beams. It does show the relationship between the camber as it changes with time, and the prestressing losses. It is in this relationship that the percentage change of the variables is so pronounced. The mean Apparent Modulus of Elasticity for beams 1 and 2 at the start of the test was found to be 3.90×10^6 psi. At 90 days the Apparent Modulus of Elasticity was 1.42×10^6 psi or 36.4% of the initial value. For beams 3 and 4 the same relative values of Modulus were 2.56×10^6 psi at zero days, 0.46×10^6 psi at 90 days, or 18% of the initial value. This shows the large difference in the change in Modulus under the two conditions. It was evident that the relative percentage prestress losses and increase in camber were cumulative in their effect on the Apparent Modulus of Elasticity.

A check on the validity of the simplified equation for the computation of the Apparent Modulus of Elasticity was made. The exact values of the Apparent Modulus, including the effects of the changing eccentricity and the bending upward of the post-tensioning bars, were calculated for all beams at zero and 90 days. Computations were also made on two of the beams at several intermediate points. The results of the exact solution and the simplified equa-

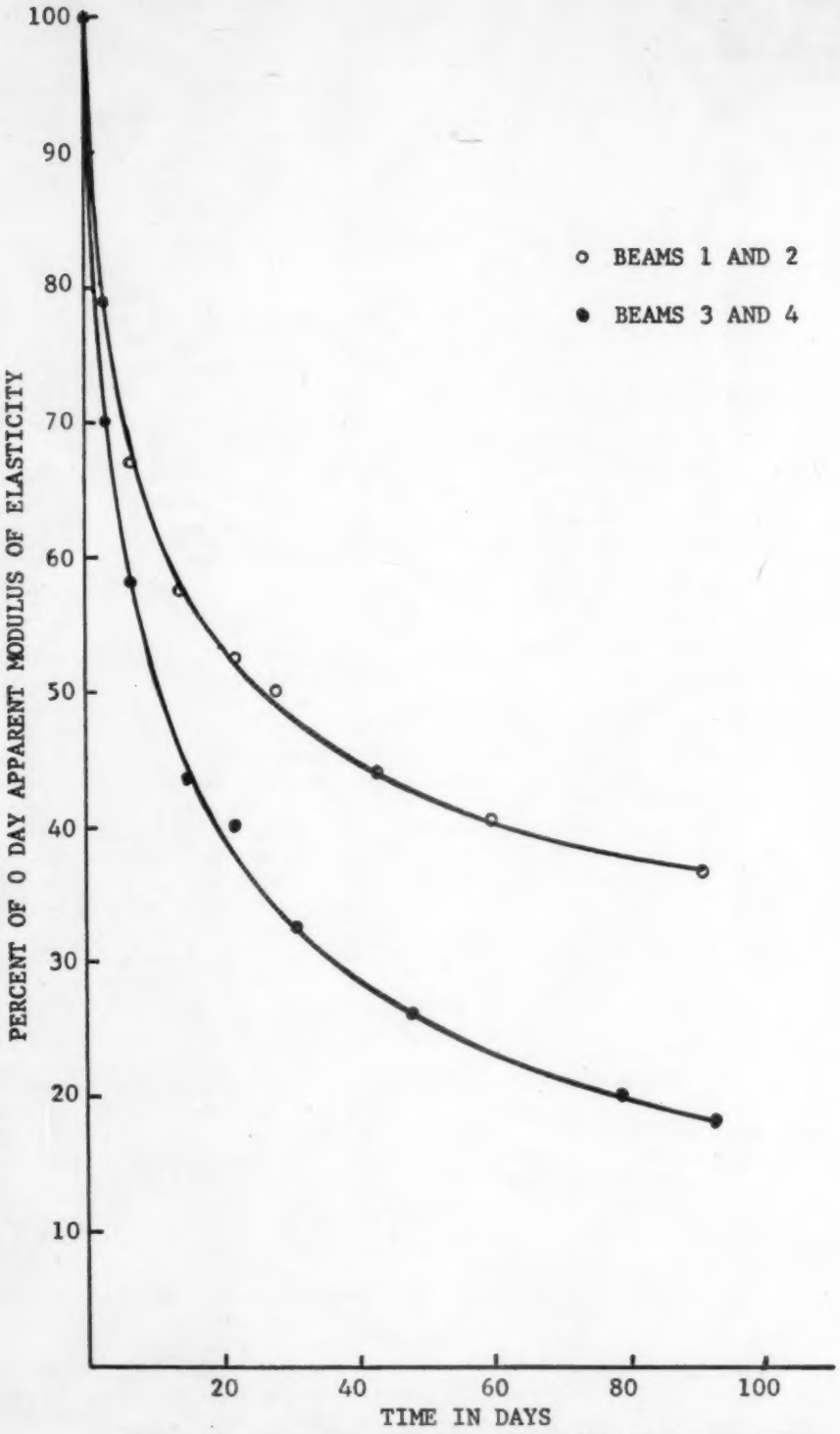
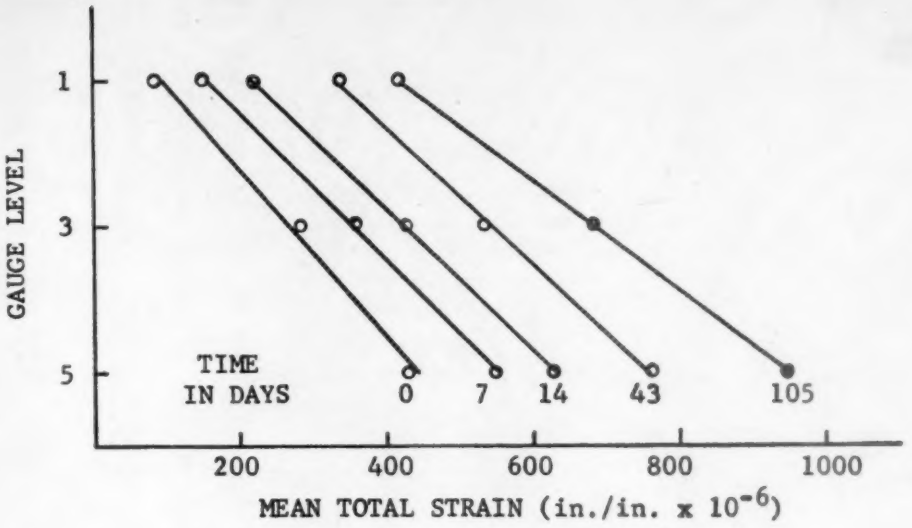


FIG. 10 - PERCENT APPARENT MODULUS OF ELASTICITY



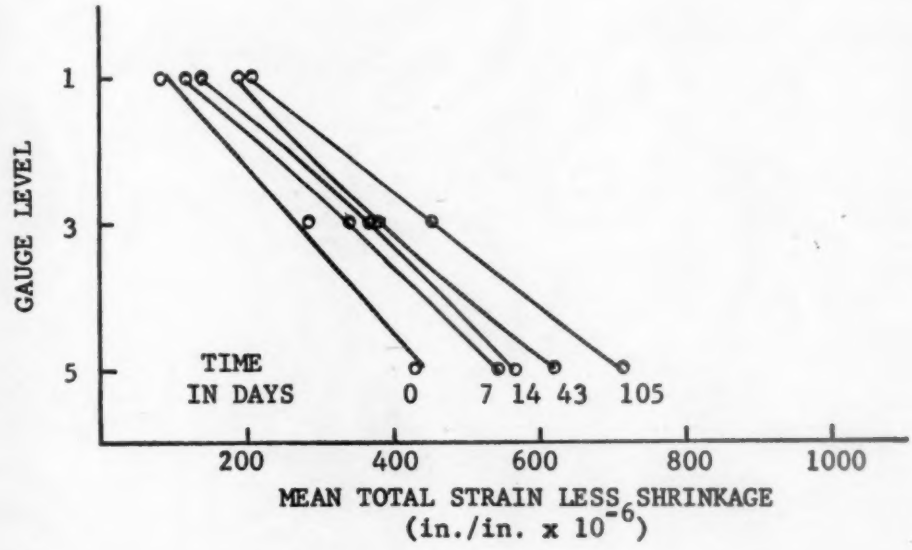


FIG. 11 - CONCRETE STRAIN DISTRIBUTION, BEAM 1, NEAR END

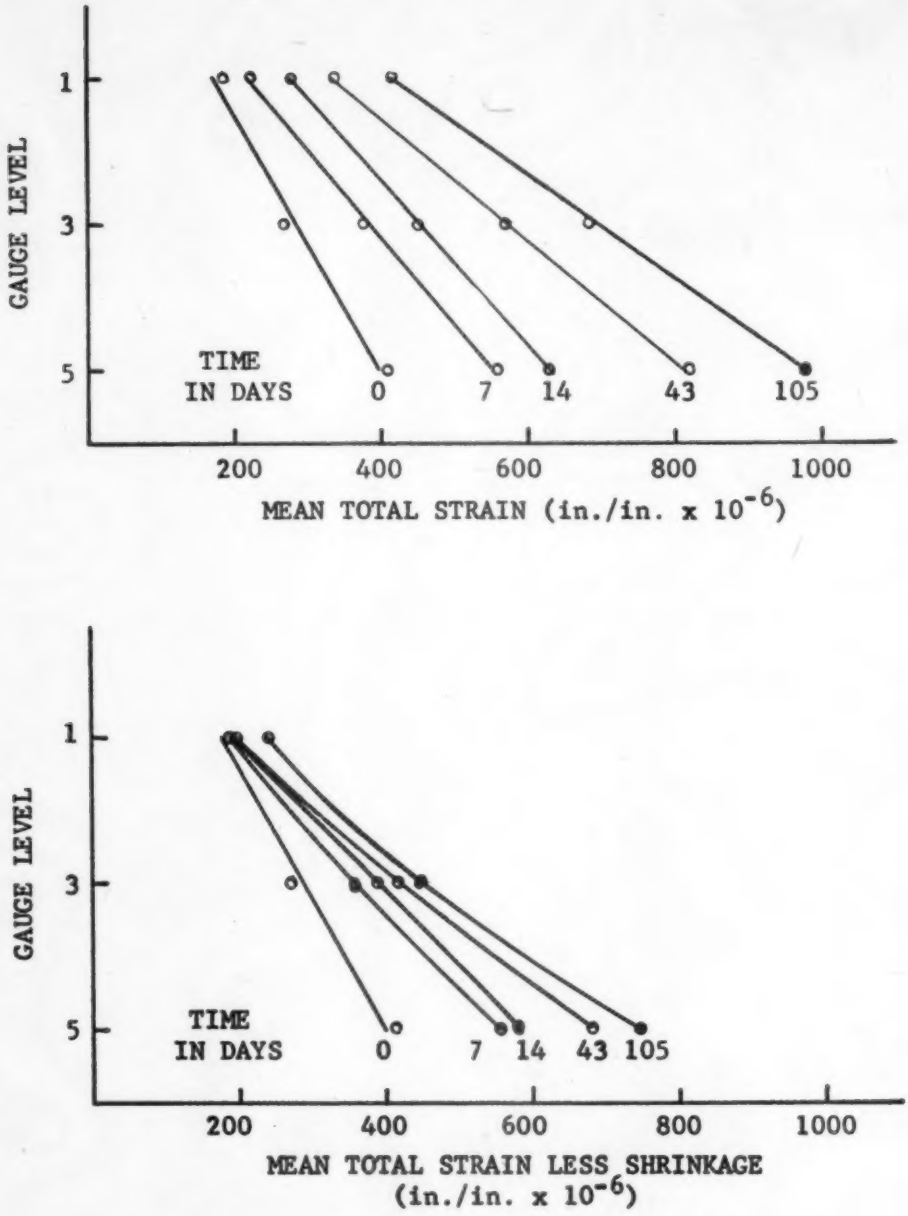


FIG 12 - CONCRETE STRAIN DISTRIBUTION, BEAM 1, CENTER LINE

tion for zero and 90 days are given in Table 1.

The curves produced by the exact solution would assume the same shape as those given by the simplified equation. The ordinates at zero days would be equal to those given in Table 1 and the exact curve would rapidly approach, almost asymptotically, the one given by the simplified solution. For the purpose of this test it appears that the simplified solution is adequate for comparison.

TABLE 1
CORRECTED APPARENT MODULUS OF ELASTICITY

Beam Number	Time of Reading days	Apparent Modulus of Elasticity psi $\times 10^6$	Corrected Apparent Modulus of Elasticity psi $\times 10^6$
1.	0	3.92	4.83
1.	90	1.47	1.52
2.	0	3.89	4.72
2.	90	1.39	1.41
3.	0	2.97	3.57
3.	90	0.54	0.73
4.	0	2.15	2.69
4.	90	0.37	0.49

The exact Apparent Modulus at zero days is of value since it represents the true Elastic Modulus at that time as no creep or shrinkage had taken place to affect the readings.

For comparison, the values of the Apparent Modulus of Elasticity of each beam were divided by the Apparent Modulus of that beam at zero days. The reason for using this representation is that the initial Apparent Modulus very nearly represents the true elastic characteristics of the beam. These values are plotted, as percentages, in Figure 10. It was found that the values determined for beams 1 and 2 were almost identical, the 90 days results giving the Apparent Modulus at 36% of the original value. Beams 3 and 4 were also nearly identical but were only 18% of the original value at 90 days. These curves point out two distinct behaviors. Even though the beams did not act alike in all respects, the percentage values of the Apparent Moduli for the beams prestressed at approximately the same stress level were almost exactly the same. Also the beams at the higher stress level had a higher percentage of Apparent Modulus than did the beams stressed at the lower stress level.

At the conclusion of the testing period (the specimens were in excess of 100 days old) the beams were loaded statically and load-deflection relationship was obtained. From this data it was possible to compute the values of the beam Modulus of Elasticity which were 2.08×10^6, 1.55×10^6, 1.73×10^6, 1.20×10^6 psi for beams 1 through 4, respectively. These values, in all cases, were higher than the Apparent Moduli and there was no consistency between two pairs of beams or the initial values computed for the Apparent Moduli.

Typical curves from the data obtained from the concrete strain readings showing the Mean Total Strain and the Mean Total Strain Less Shrinkage are given in Figures 11 and 12. These curves show the strain as the abscissa and location of the gauge as the ordinate for Beam 1. The mean value refers to the average of both sides of the beam.

As expected the extension of the Mean Total Strain curves at the zero day readings show zero strain in the top of the beam at the ends. This indicates that the steel was located, as computed, at the lower kern point. This is not true in the center of the beams where there were dead load stresses due to the weight of the beam after prestressing. It can be seen that in nearly all cases the Mean Total Strain curves were actually straight lines. This is in accord with the known test data which shows that plain surfaces of beams remain plain after bending.

The curves for the Mean Total Strain Less Shrinkage do not give straight lines after the zero day reading. This is due primarily to the uneven rate of shrinkage at the various levels in the beam. The Mean Total Strain Less Shrinkage actually represents the elastic strain due to post-tensioning plus the creep. It is apparent from the relationship between these two curves that where the shrinkage was great, the creep was less than normal. This would necessarily be true if the curve of the Mean Total Strain at a section is to be a straight line. In other words, it appears that the

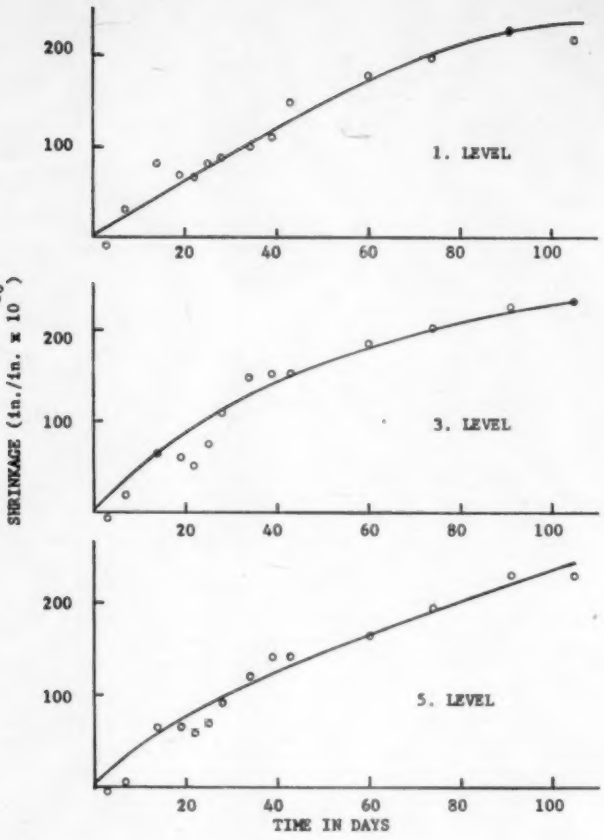
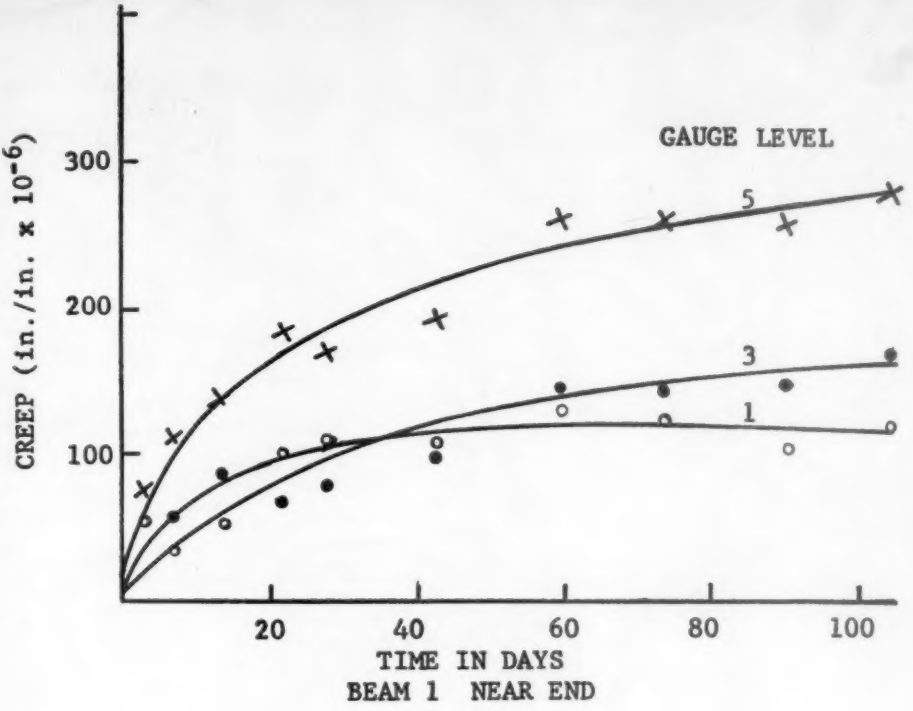


FIG. 13 - CONCRETE SHRINKAGE CURVES

shrinkage plus the creep at any given time can be expressed as the sum of two terms. These are the shrinkage at the neutral axis of the beam, plus some function of time multiplied by the distance from the neutral axis to any point on the section. The variable would have to represent the creep characteristics and the different rate of shrinkage of the section. The variable would be a constant for any given time, thus giving the equation of a straight line.

The shrinkage curves used in these calculations are given in Figure 13. It can be seen that at the end of the three months covered by this testing period, all of the curves had reached about the same level. The only major difference was the rate of shrinkage. In this case level 3 showed the fastest rate of shrinkage in the early portion of the test but leveled off sufficiently for the concrete at levels 1 and 5 to catch up. This difference in rate of shrinkage is responsible for the curving of the Mean Total Strain Less Shrinkage curves.

Creep curves were plotted for all locations. The curves are given in Figures 14 through 19. All of the curves appeared to follow a normal pattern with the creep at the 1 level being the least and the creep at the 5 level the greatest. Probably the most significant behavior indicated by this set of curves is the fact that there is no marked difference in the creep values determined at respective points for the two differently stressed pairs of beams. There was no marked difference in creep or the rate of creep at ali corresponding locations. This suggests that additional information and testing is necessary to substantiate these results which



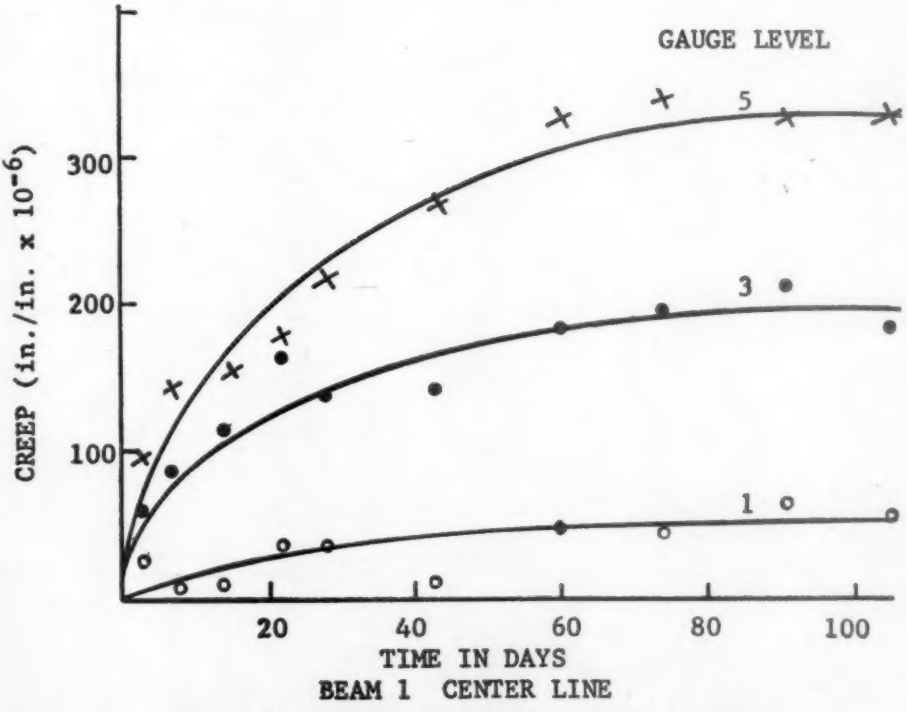


FIG. 14 - CONCRETE CREEP CURVES, BEAM 1

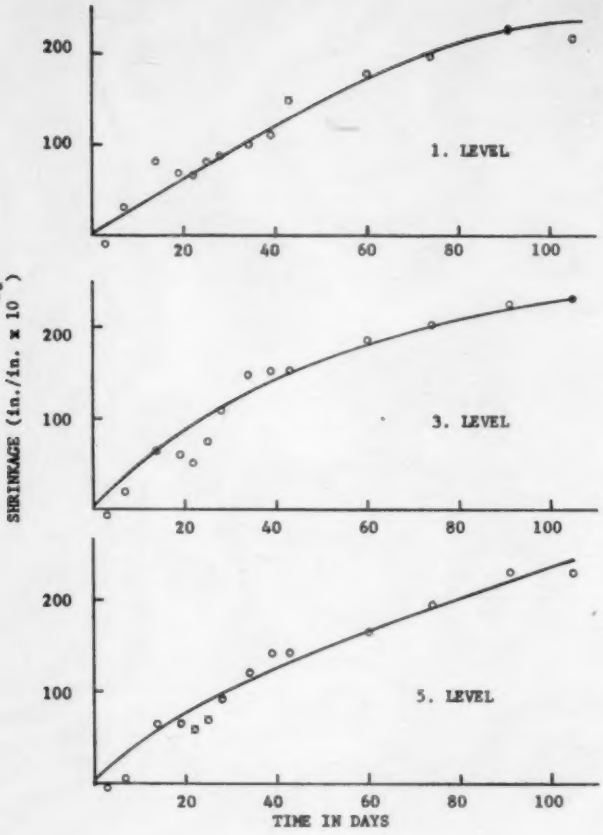


FIG. 13 - CONCRETE SHRINKAGE CURVES

shrinkage plus the creep at any given time can be expressed as the sum of two terms. These are the shrinkage at the neutral axis of the beam, plus some function of time multiplied by the distance from the neutral axis to any point on the section. The variable would have to represent the creep characteristics and the different rate of shrinkage of the section. The variable would be a constant for any given time, thus giving the equation of a straight line.

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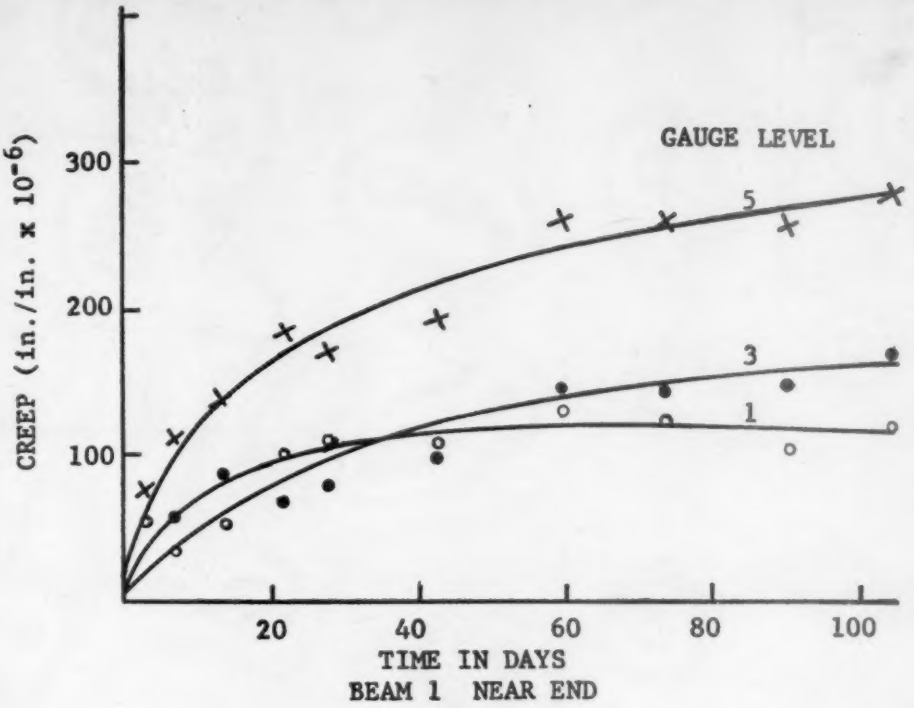


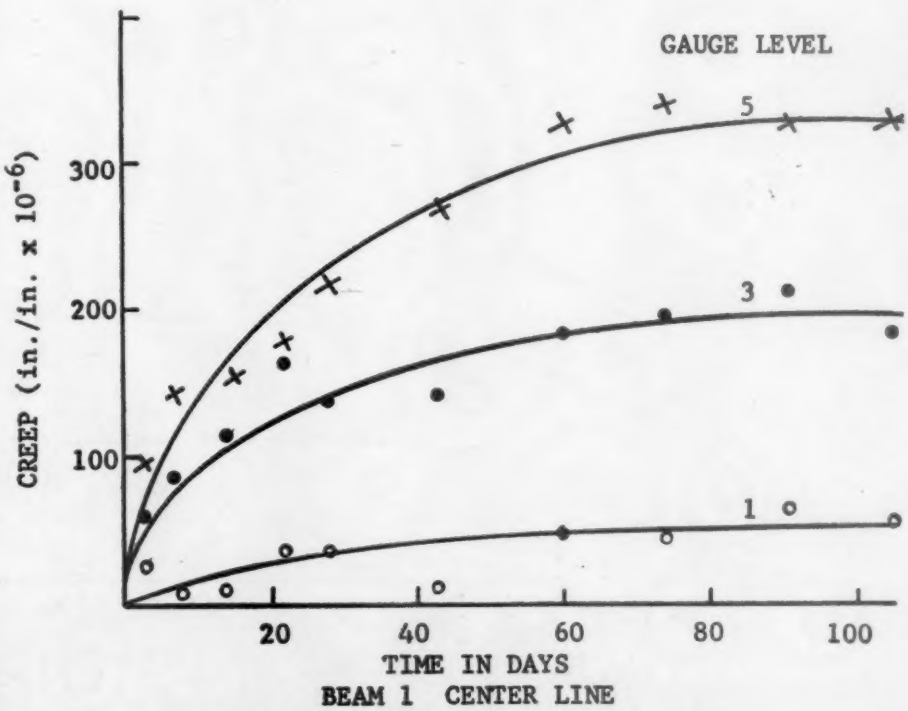
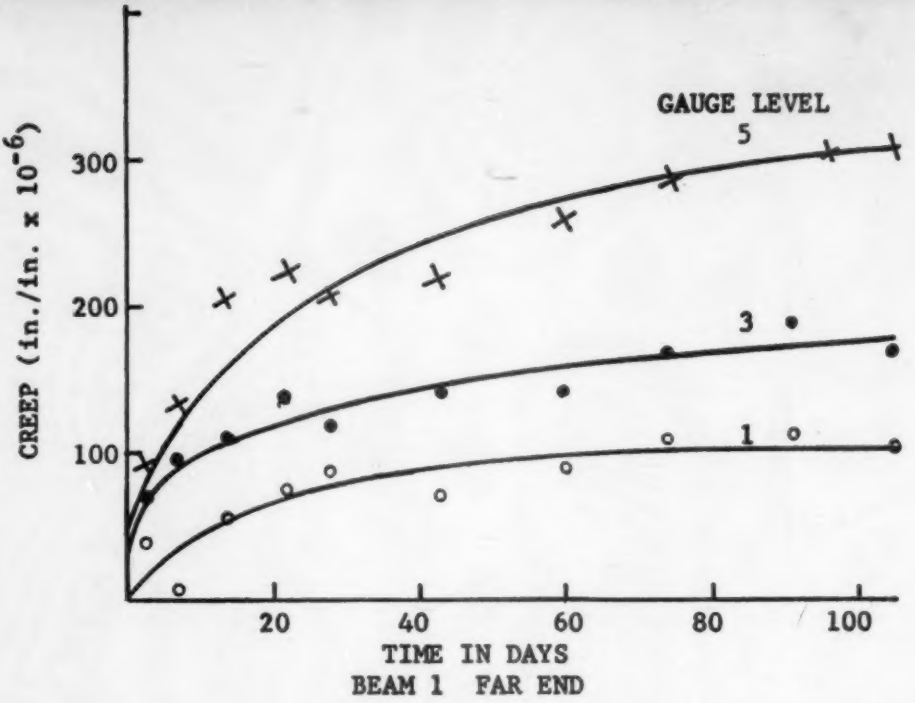


FIG. 14 - CONCRETE CREEP CURVES, BEAM 1



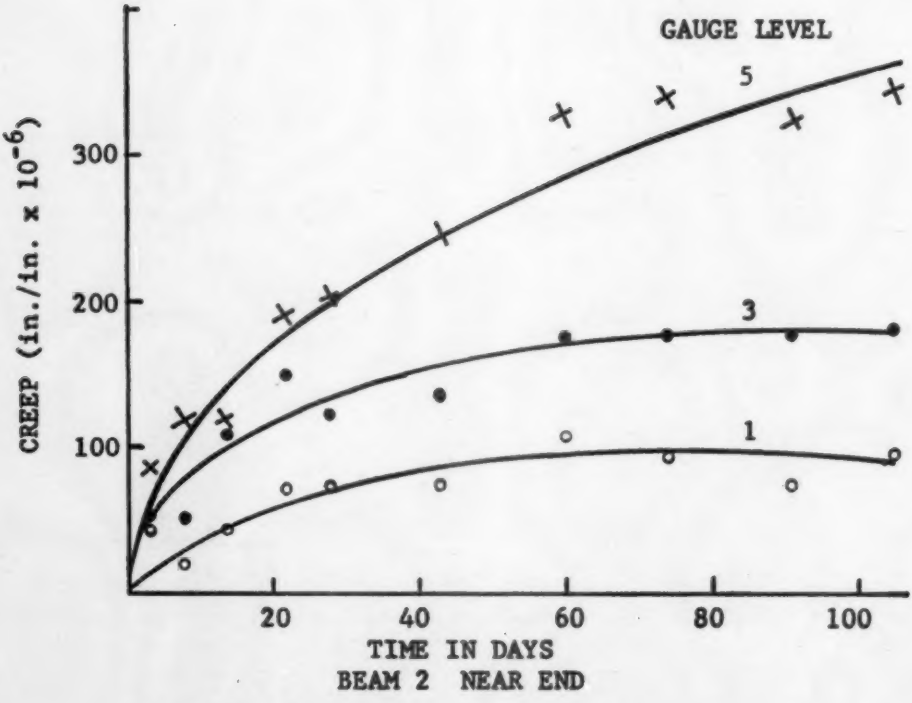
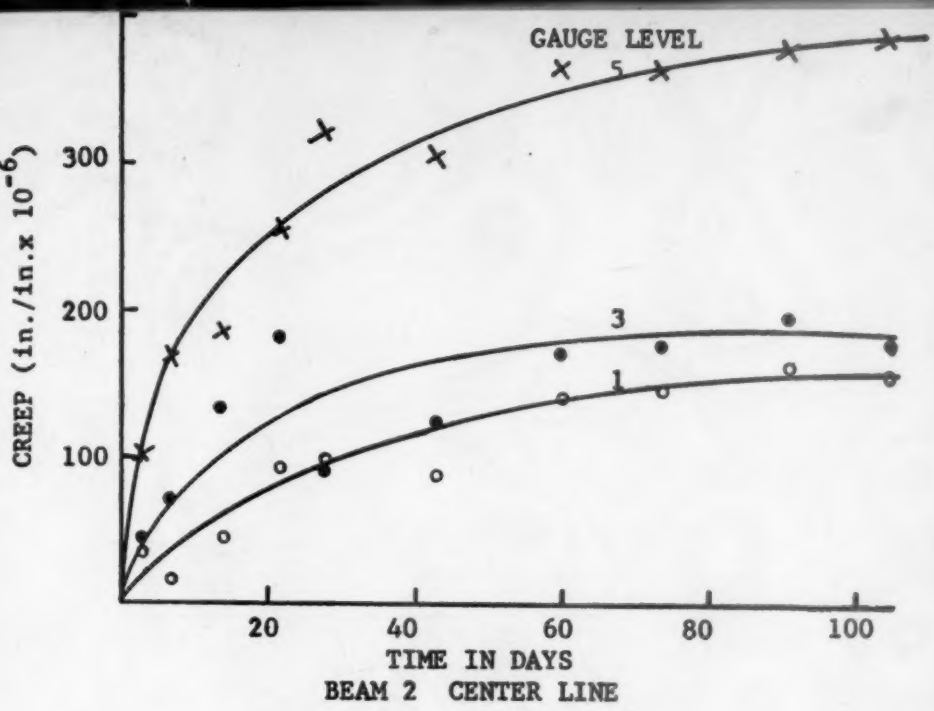


FIG 15 - CONCRETE CREEP CURVES, BEAMS 1 AND 2



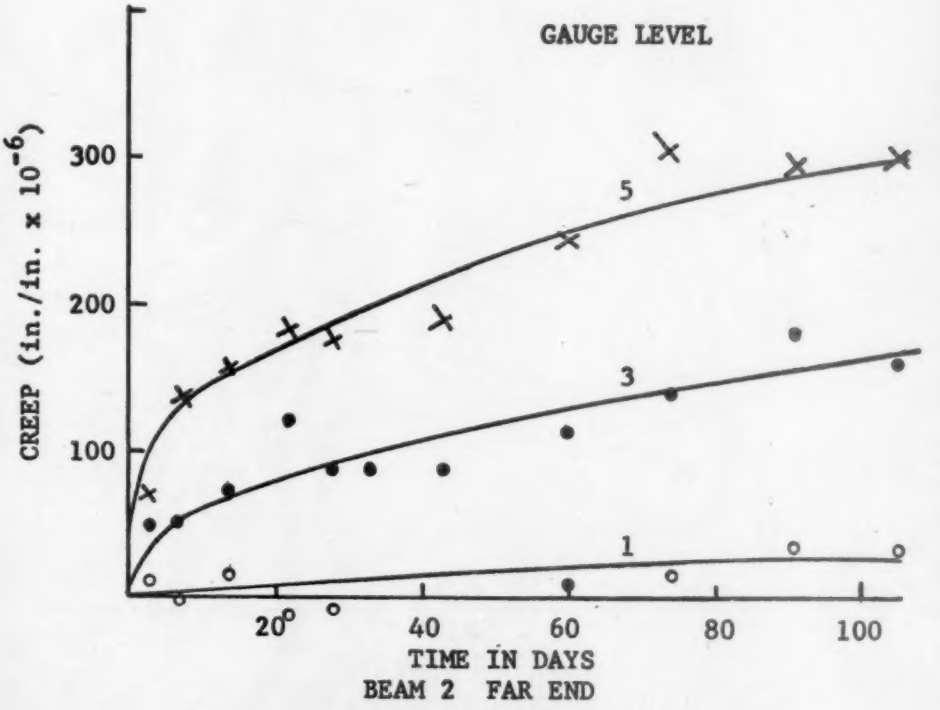
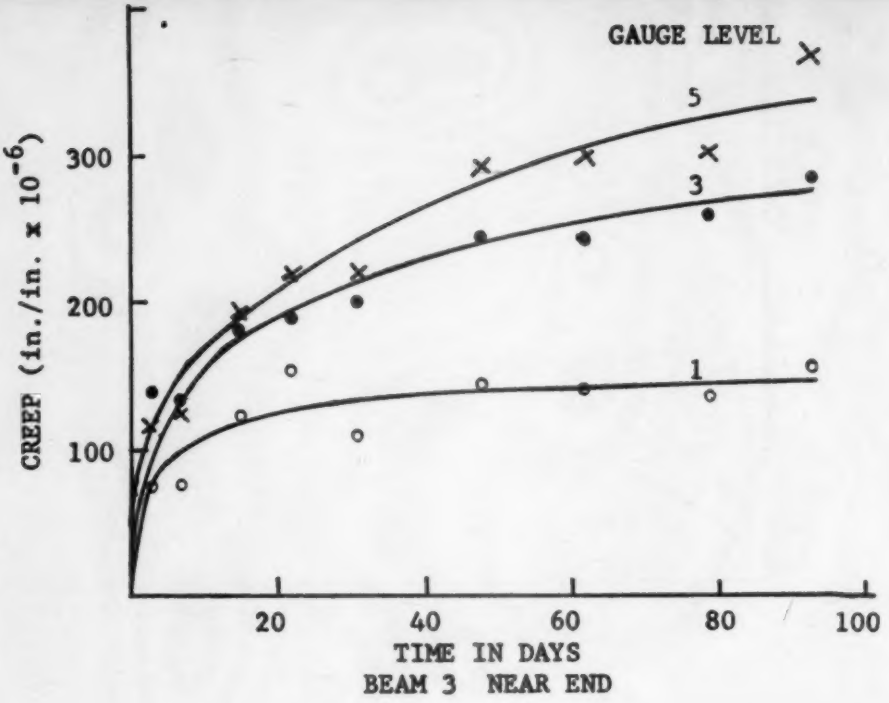


FIG. 16 - CONCRETE CREEP CURVES, BEAM 2



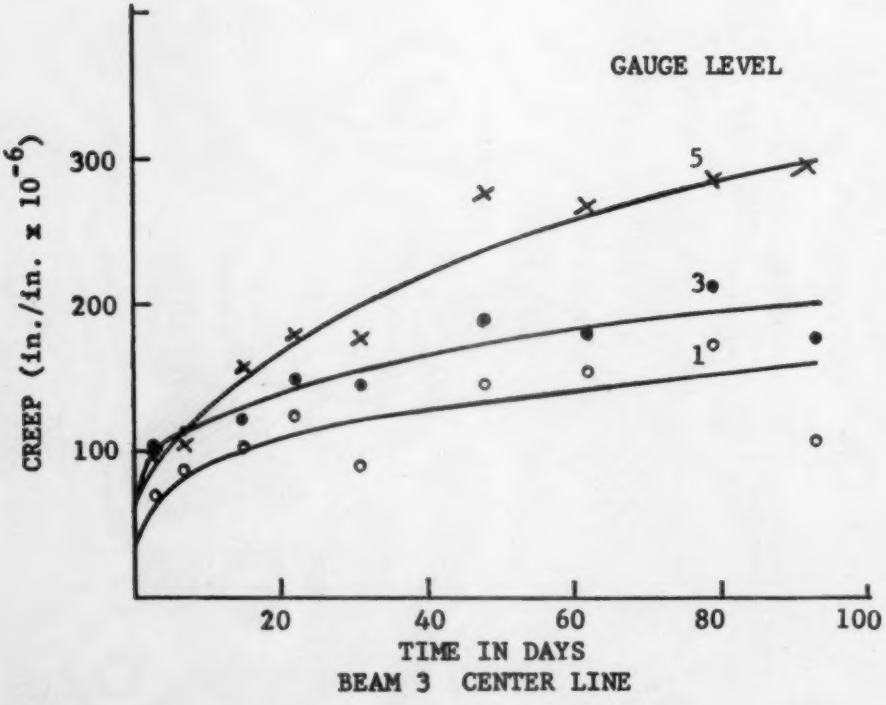
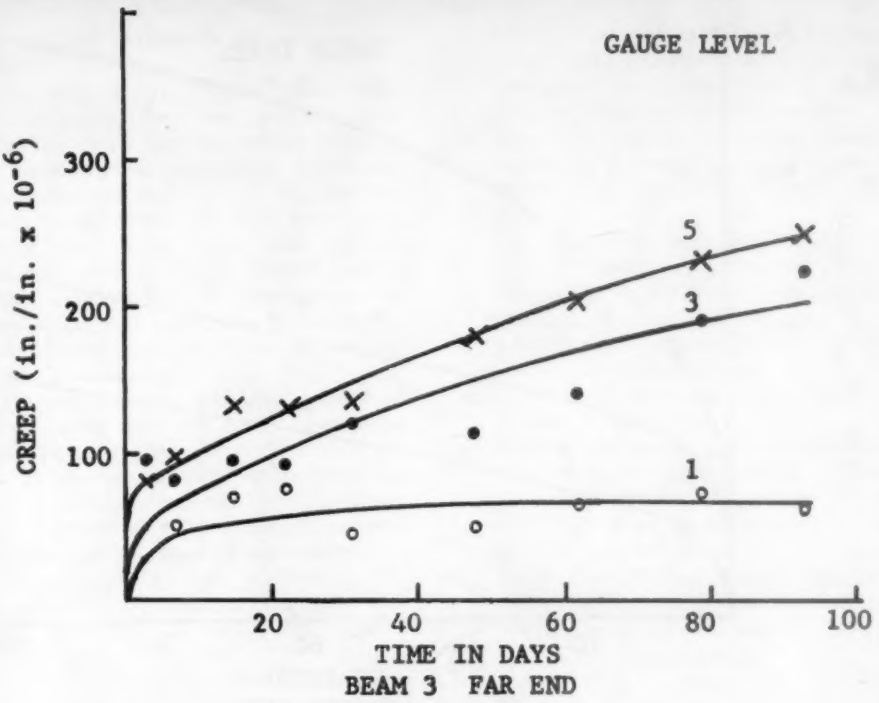


FIG. 17 - CONCRETE CREEP CURVES, BEAM 3



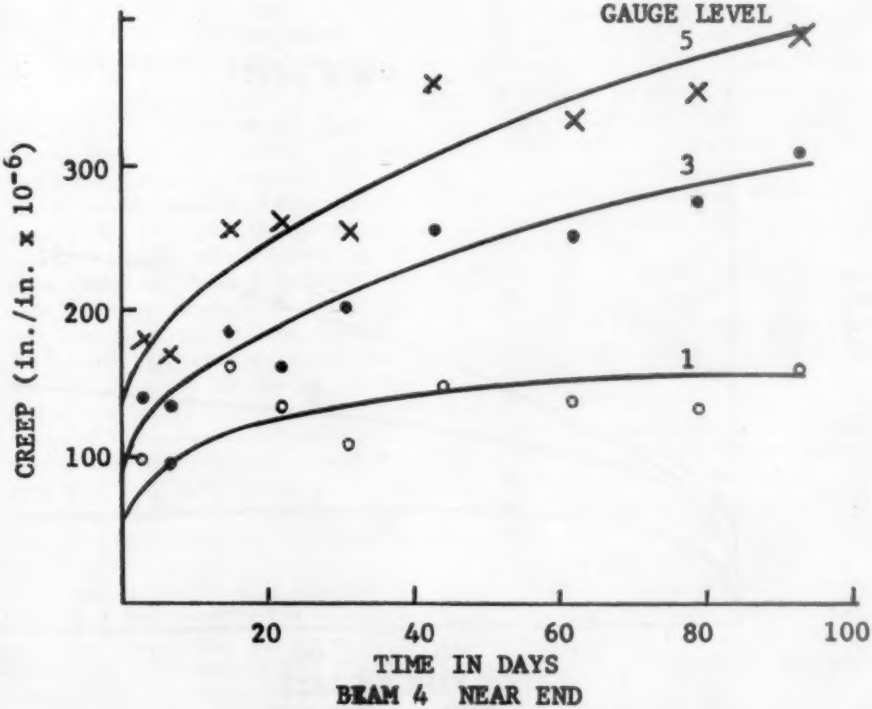
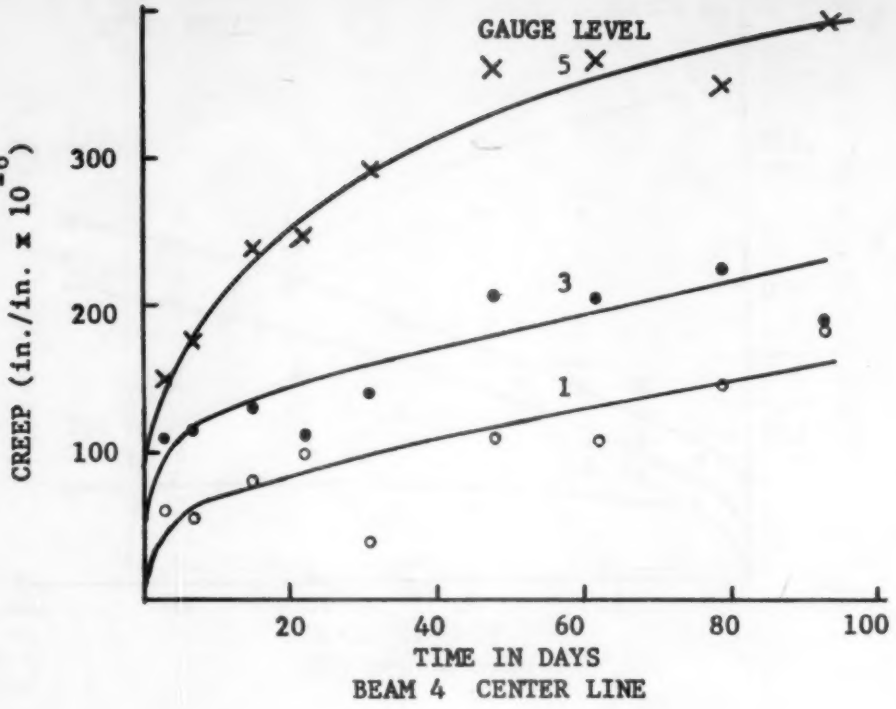


FIG. 18 - CONCRETE CREEP CURVES, BEAMS 3 AND 4



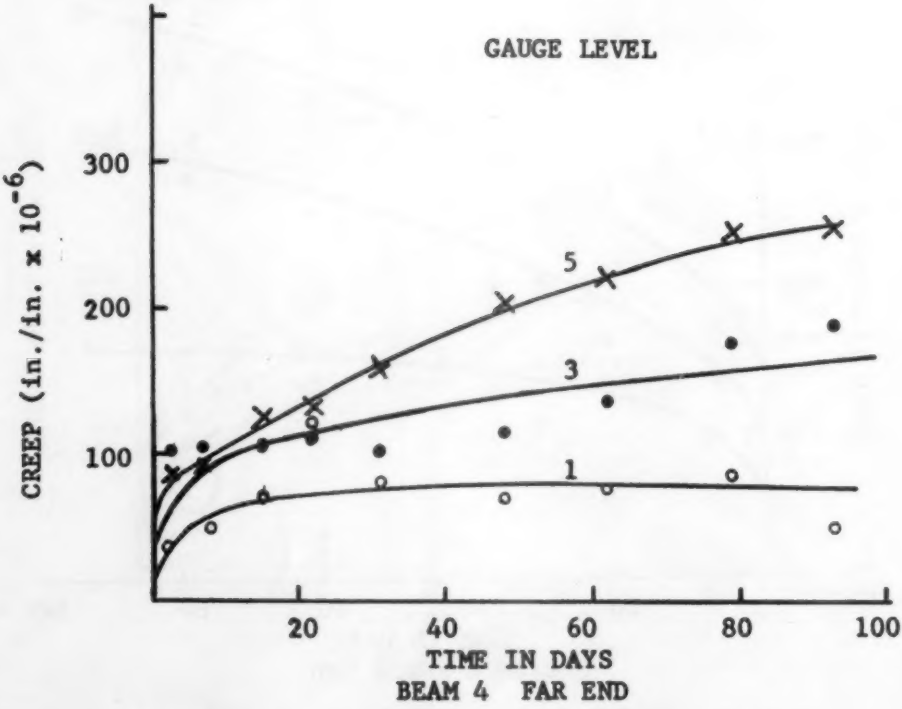


FIG. 19 - CONCRETE CREEP CURVES, BEAM 4

appear to be contrary to the accepted behavior.

The creep at level 5 varied from 250 to 400 microinches per inch for the beam ends. The elastic shortening of the beams at the same locations varied from 300 to 480 microinches per inch. Considering that the testing period covers only three months, the figures agree quite closely with the assumptions made for design. The ACI-ASCE Joint Committee 323 (5) assumes that the creep strain will vary from 100 to 300% of the elastic strain depending on the surrounding conditions.

The magnitude of shrinkage determined agrees closely with the design assumptions generally used. The value determined by test at 90 days was 225 microinches per inch. The values generally used in design vary from 200 to 300 microinches per inch depending on the surrounding conditions.

The suggested Modulus of Elasticity did not agree with the test data. The ACI-ASCE Joint Committee 323 recommends using;

$$E = 1{,}800{,}000 + 500f'_c.$$

For the concrete used in the test, this value becomes 5.2×10^6 psi. The Modulus of Elasticity in flexure, determined experimentally, varied from a high of 4.8×10^6 psi at zero days to a low of 1.2×10^6 psi at the end of the testing period. The Modulus suggested by Committee 323 is quite high compared to the experimental data.

CONCLUSIONS

From the test results of the particular specimens used in this program the following conclusions were reached:

1. The Apparent Modulus of Elasticity decreased with time. This change was definitely a function of the creep. In all cases, except for beams 1 and 2 at zero days, the Apparent Modulus of Elasticity was less than the cylinder Modulus. It was also less than the beam Modulus of Elasticity in flexure at the end of the testing period. The average corrected value for the Apparent Modulus of Elasticity for beams 1 and 2 at zero days was 4.78×10^6 psi. This agrees very closely with the cylinder Modulus of 4.70×10^6 psi determined at zero days.
2. The Apparent Modulus of Elasticity of the beams prestressed to 0.40 f'_{ci} was at all times higher than the value determined for the beams at 0.25 f'_{ci}. Expressed as a percentage, beams 3 and 4 had an Apparent Modulus of Elasticity at zero days equal to 66% of the value determined for beams 1 and 2. At 90 days this ratio had dropped to 30%.
3. There does not appear to be a definite relationship between the Apparent Modulus of Elasticity and the cylinder properties.
4. The creep strain plus the shrinkage was linear across the cross section.
5. The creep in the concrete at different locations of each beam appeared to be a function of both the stress magnitude and the relative shrinkage.
6. Paradoxically there was no marked difference in the creep or rate of creep readings taken at corresponding locations in beams prestressed to different stress levels (maximums of 0.40 f'_{ci} and 0.25 f'_{ci}). This was evidenced by the camber measurements of all beams which showed approximately the same amount of increase of 0.40 in. in camber during the 90-day test period.

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5. ACI-ASCE Joint Committee 323. "Tentative Recommendations for Prestressed Concrete," Portland Cement Association, Chicago. 1958.

ACKNOWLEDGMENT

These tests were conducted as part of the research program sponsored by the Florida State Road Department. Acknowledgment is made to Flexico Products, Inc. and Stressteel Corporation as donors of the conduit and post-tensioning bars used in this work.

Prestressed Piles over 200 feet long

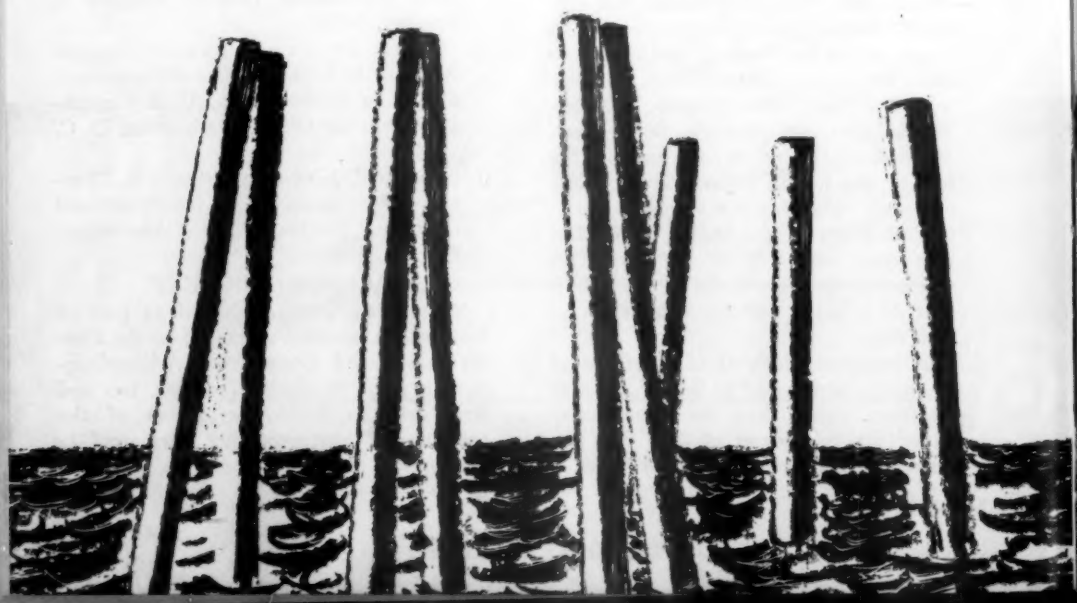
BY MYERS VAN BUREN

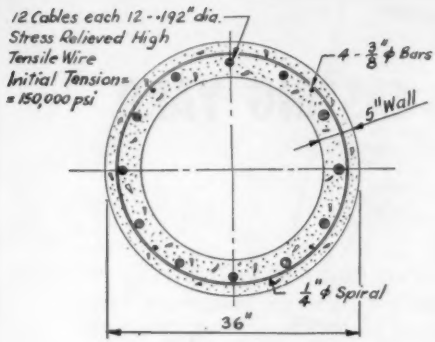
Prestressed Products Engineer
Raymond International, Inc.
New York, N. Y.

Prestressed piles are becoming longer. During the past two years hundreds of piles over 180 feet long have been driven in Lake Maracaibo, Venezuela for various oil drilling and production platforms.

The longest piles have been 208 feet. Piles of the 208 foot length were made up of 12 centrifugally cast sections each 16 feet long, 36 inches outside diameter and with a 5 inch wall. Post-tensioning was with 12 cables each of 12 # 6 wires stressed to 150,000 p.s.i. The post-tensioned wires were grout bonded with no permanent anchorage.

Piles of this size and length are driven plumb and on batters up to 1 on 3½. Handling and driving has presented no particular problem. Two types of driver have been used. One was a special batter leader driver and the other a standard whirley, barge mounted, with special batter leads handled by the crane.





PILE CROSS SECTION

These long piles have been handled with a three point pickup without overstressing the concrete. Single acting hammers with ram weights of 12,500, 15,000 and 17,500 pounds have been used. Their corresponding energies are 40,625, 48,750 and 56,875 foot-pounds. The piles have been installed both open and closed end, depending upon conditions. The 208 foot units were closed end and were driven with the 56,875 foot-pound hammer.

The piles, both plumb and on batters of 1 on 3½, have been installed in water depths up to 105 feet without bracing. The axial load on the vertical piles exceeds 250 T under extreme design conditions. The 36″ hollow prestressed cylinder satisfactorily meets the requirements of design under these conditions.

The only length limitation on the hollow prestressed cylinder is the capacity of the installation equipment. It is expected that much longer piles will be used within a few years.

The piles described in this article were manufactured in the plant of Concretos Industriales C. A. at Potrerito, Venezuela, under license from the Cen-Vi-Ro Pipe Corporation for the section casting methods and Raymond International Inc. for the pile manufacturing methods. Installation was by Raymond-Brown & Root C. A., Maracaibo, Venezuela, jointly owned by Raymond International Inc., New York and Brown & Root, Inc., Houston Texas.

ADVENTURES IN A CASTING YARD

(A Homily for Producers Only)

by

MARTIN P. KORN

"Long distance calling PRESTRESSED CONCRETE INSTITUTE . . ."

"This is Mr._____. I am going into the prestressing business. Do you have any publication on how to lay out a plant with beds and equipment?"

"No," I replied, "we always recommend the employment of experienced and specially qualified Consulting Engineers for that purpose."

"Oh, I don't need that" came from a cocky voice. " I will lay out the yard myself. I know just how I want it to be. But I thought you could give me some extra advice."

"Are you experienced at plant design?" I asked.

"No, but I worked three years for _____ who has your Plant Standards. I know all about it. All I want is, er, you know what I mean — Maybe you got standards for casting yards . . ."

Thus, almost verbatim, a story was told which I had heard so often before. Just another "do it yourself" case so prevalent in the prestressed concrete industry, where mistakes are copied from other yards, funds buried in dead weight, costly initial investments made by non-technical, non-qualified prospectors.

The aim here is to call out to all potential and established manufacturers of prestressed concrete to stop in place, look and listen. And this JOURNAL will pick at random one example of a typical live case study which I hope will stop the march of irresponsible ventures minus technology.

Subject: **Case study of two beds of equal length, width and jacking capacity.**

Design by Mr. Experienced Consulting Engineer: 96 cu. yd. of concrete

Design by Mr. NO Experience .....448 cu. yd. of concrete

ABUTMENT DESIGN

Mr. C. E.: Light, simple and easily expandable.

Mr. NO: Heavy, cumbersome. Cannot be expanded without great expense.

OVERALL PLANT PLANNING

Mr. C. E.: Design—simple, logical, materials handling schemes which allow no interference between stripping, set-up, pouring, storage and loading out.

Mr. NO. Not designed, haphazard growth with no planning with regard to handling and storage, resulting in great interference between operations which must be carried on simultaneously.

Thus you meet a typical Mr. NO who builds prestressing plants by phone, hit or miss. And the sad fact is that too many casting yards were planned by Mr. NO's than were planned by experienced engineers.

Does this case study point up the need for reappraisal of your plant?

EDITOR'S NOTE:

Not much more than two years have passed since the American Association of State Highway Officials and the Prestressed Concrete Institute joined hands, minds and experience to produce "Standard Prestressed Concrete Beams for Highway Bridge Spans of 30 ft. to 100 ft." Today, bridges built to those standards are to be found in 27 states. Of added significance is the news that prestressed concrete has been used in 16 other states, making a total of 43 states where you drive over prestressed concrete bridges. What a phenomenal record for a new construction material introduced here from Europe only ten years ago and now is just about covering the country!

The survey which follows amplifies those astonishing figures. P.C.I. is grateful to the Freyssinet Company of New York for making them available to us.

As you opened this issue of the JOURNAL, you were greeted by our guest Editorial writer, Mr. Glenn S. Paxson, Chairman, Committee of Bridges and Structures, American Association of State Highway Officials, who had a few kind and complimentary words for us. And nearing the end of this JOURNAL, you will meet the genial chairman of the Technical Activities Committee and a galaxy of PCI Committees and joint AASHO-PCI committees on the march, shaping new destinies and greater horizons for the future of prestressed concrete. This parade of technical experts passing in review will converge in a glittering assembly at the forthcoming PCI Convention at Miami Beach — a dramatic national spectacle of progress depicting "PCI AT WORK."—M.P.K.

SURVEY of USE of AASHO-PCI STANDARDS for HIGHWAY BRIDGES

STATE	Use	TYPE USED				
	Prestressed Bridges	AASHO PCI	State Standard	B.P.R.	Box Beams	Contractors Design
Alaska	Yes					
Arizona	Yes	X				
Arkansas	Yes	X				
California	Yes		X			
Colorado	Yes	X				
Connecticut	Yes					X
Delaware	Yes				X	
Florida	Yes	X				
Georgia	Yes		X			
Hawaii	Yes	X				
Idaho	Yes	X				
Illinois	Yes		X		X	
Indiana	Yes			X	X	
Iowa	Yes		X			
Kentucky	Yes					
Louisiana	Yes	X				
Maine	No					
Massachusetts	Yes		X			
Michigan	Yes	X				
Minnesota	Yes	X				
Missouri	Yes	X				
Mississippi	Yes	X				
Montana	Yes		X			
Maryland	Yes					X
Nebraska	Yes	X				
Nevada	Yes		X			
New Hampshire	No					
New Jersey	Yes	X				
New Mexico	Yes	X				
New York	Yes					X
North Carolina	Yes	X				
North Dakota	Yes	X				
Ohio	Yes	X				
Oklahoma	Yes	X				
Oregon	Yes	X				
Pennsylvania	Yes				X	
Puerto Rico	Yes	X				
Rhode Island	Yes	X				
South Carolina	Yes	X				
South Dakota	Yes	X				
Tennessee	Yes	X				
Texas	Yes		X			
Utah	Yes	X				
Vermont	No					
Virginia	Yes	X				
Washington	Yes		X			
West Virginia	Yes					X
Wisconsin	Yes	X				
Wyoming	No					

HIGH SPEED FATIGUE TESTS
on
SMALL SPECIMENS
of
PLAIN CONCRETE

(A METHOD FOR DETERMINING THE FATIGUE STRENGTH OF PRESTRESSED CONCRETE BEAMS)

●

By

BASIL M. ASSIMACOPOULOS
Research Assistant

ROBERT F. WARNER
Research Associate

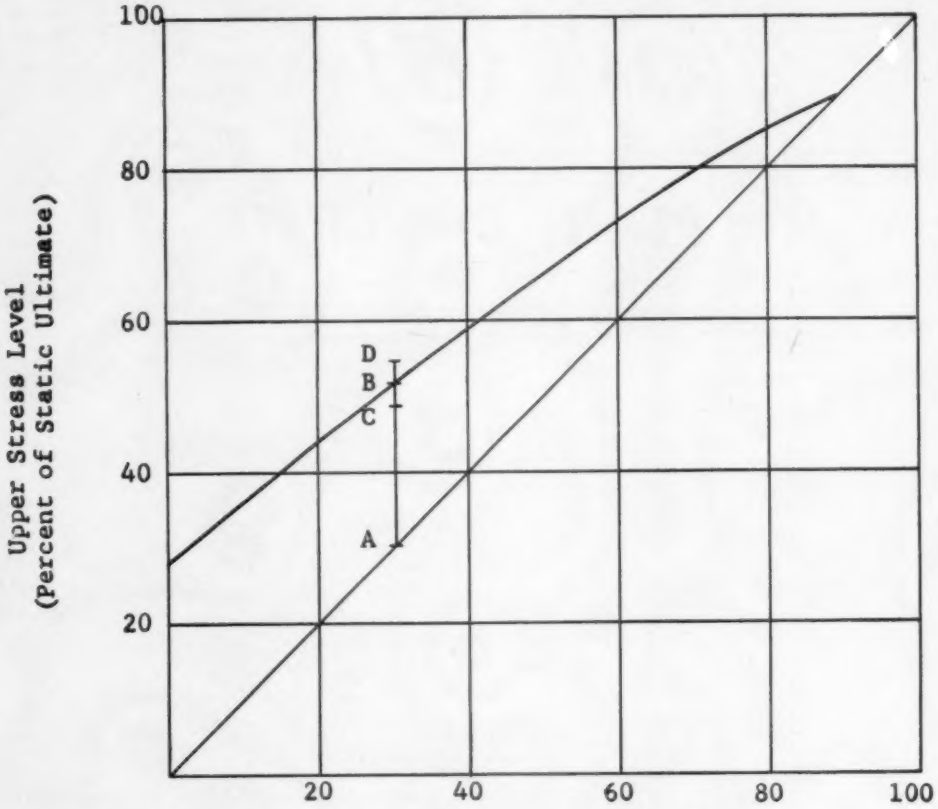
CARL E. EKBERG, JR.
Associate Professor of Civil Engineering
Chairman, Structural Concrete Division

Institute of Research
LEHIGH UNIVERSITY
Bethlehem, Pensylvania

SYNOPSIS

A brief description is given of the method of determining the fatigue strength of prestressed concrete beams, based on separate fatigue failure envelopes for the concrete and high strength steel. The importance of fatigue tests on specimens of plain concrete and steel is thus emphasized.

The main portion of the paper describes a series of pilot fatigue tests on 2 x 4 inch cylinders of plain concrete which were conducted at speeds of 9000 and 500 cycles



Stress Range	Number of Cycles to Failure
A B	1,000,000
A C	more than 1,000,000
A D	less than 1,000,000

Fig. 1 - FATIGUE FAILURE ENVELOPE FOR PRESTRESSING STEEL

per minute. The results of the tests are presented as S-N curves and as fatigue failure envelopes for 500,000, 1,000,000 and 2,000,000 cycles.

INTRODUCTION

Fatigue Resistance of Prestressed Concrete Beams

In 1958 a method was proposed for predicting the flexural fatigue resistance of prestressed concrete beams.(1) The method utilizes the fatigue-failure envelopes of the component steel and concrete, and the stress-moment relations for the critical section of the beam.

A fatigue-failure envelope is a graphical means of representing those ranges of stress which will cause fatigue failure in a given material, if repeated one million times. The stress variation can be defined by the minimum value and the maximum value; thus, for any given minimum stress level the maximum stress level of the critical load cycle may be found from the fatigue failure envelope. An example of a fatigue failure envelope for high tensile steel is contained in Fig. 1. This envelope has been checked by Laboratory tests at Lehigh University on steel strands of the type used for prestressed concrete.(2)

The stress-moment relations indicate the state of stress existing in the tension reinforcement and at the extreme fibers of a cross section of a beam when a moment, between zero and the static ultimate, is applied to the section. A means of obtaining the stress-moment relations for a prestressed concrete member is described in Reference 3.

As a matter of convenience, the fatigue failure envelopes and the stress-moment relations are presented in non-dimensional form with moment as a percentage of the static ultimate moment of the section and stress as a percentage of the corresponding static ultimate stress of the material.

Fatigue failure of a beam may occur either by a tensile fatigue failure of the longitudinal steel or by a compressive fatigue failure in the top fibers of the concrete. The critical ranges of loading which will cause fatigue failure after one million cycles may thus be determined by using the stress-moment relations to convert the critical stress ranges, given by the fatigue envelopes, into critical moment ranges. For a more complete explanation of the theory, the reader is referred to Reference 1.

Satisfactory correlation has been obtained with the test data reported by numerous investigators, such as Eney and Knudsen (4), Ozell and Ardmann (5), Inamata (6), Slutter and Ekberg (7) and others; however in all of the cases studied, failure of the beams occurred by fatigue in the steel. From the point of view of fatigue failure, such beams may be termed under-reinforced, an over-reinforced beam being one in which failure is associated with fatigue of the concrete top fibers.

Fatigue Failure in Over-reinforced Beams

A diagram for predicting the fatigue strength of a typical over-reinforced prestressed concrete beam is shown in Fig. 2. It consists of stress-moment curves in the central portion of the figure with the steel and concrete fatigue failure envelopes at the left side and right side respectively. Two ranges of loading, AB and CD, are shown which will cause failure in the beam by fatigue in the concrete. In each case the minimum moment might be considered to correspond to a dead load, and the maximum moment to the dead load plus live load.

Referring to Fig. 2, a cycle of loading AB causes a stress cycle of A′B′ in the steel reinforcement and A″B″ in the concrete top fiber. Whereas A′B′, a stress range of 8 percent, is about half of the range required to cause failure in the steel, A″B″, approximately 50 percent, touches the concrete fatigue envelope and is sufficient to induce failure in the concrete after one million repetitions.

The second example corresponds to a higher dead load but smaller live load. Again, the stress range C′D′ does not seriously affect the steel whereas the concrete stress range C″D″ matches the fatigue envelope and is critical.

Importance of Fatigue Tests of Plain Concrete and Steel Specimens

The fatigue theory outlined above indicates the necessity of studying the fatigue properties of the component steel and concrete in order to determine the fatigue strength of the composite member. The authors believe that the practical importance of the failure envelopes cannot be over-emphasized as tools for determining fatigue strength of prestressed concrete members. Current

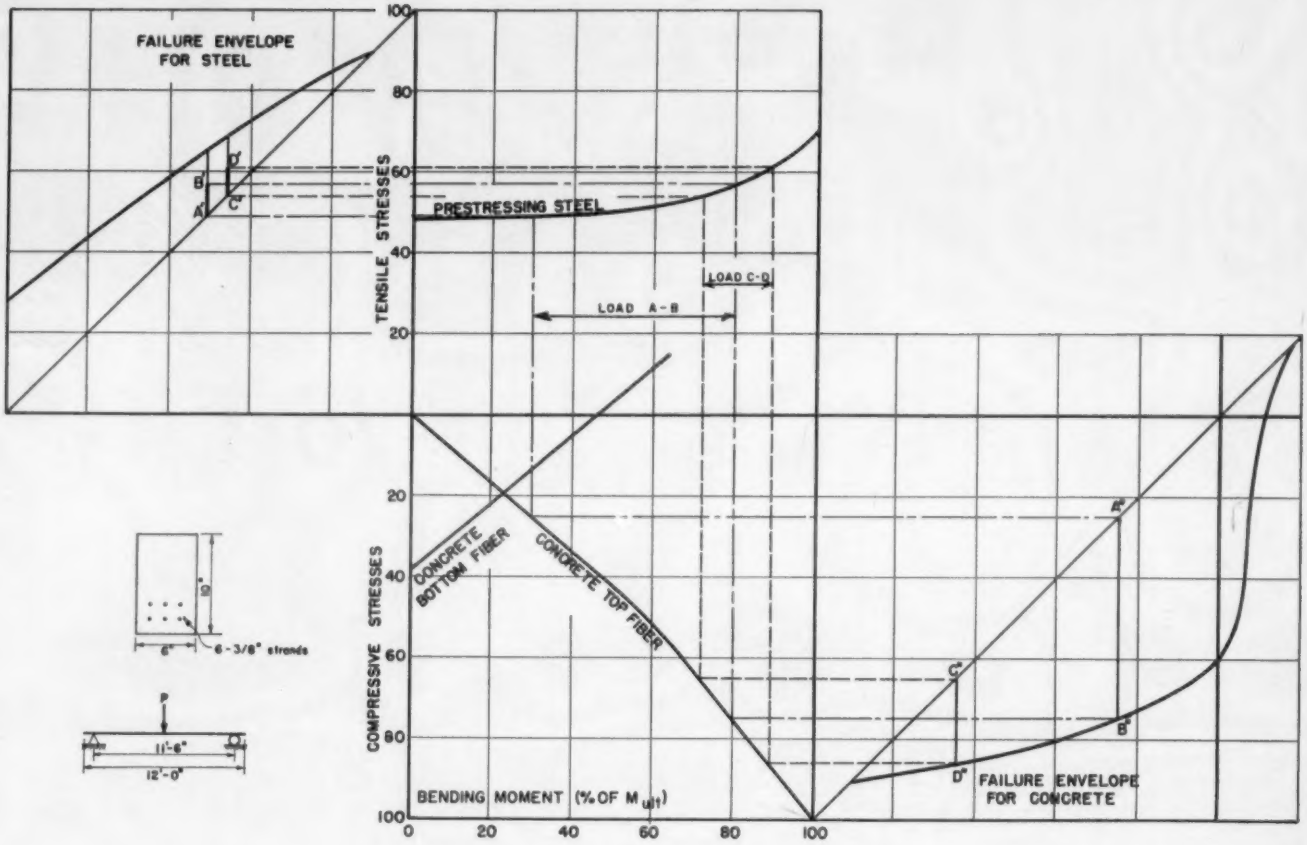


FIGURE 2. COMBINED DIAGRAM FOR OVER-REINFORCED BEAM

work at Lehigh University, extending the theory of fatigue failure to deal with beams subjected to varied patterns of loading such as those which occur during the lifetime of a bridge girder, has further emphasized the importance of fatigue tests on plain concrete and steel specimens.

TEST PROGRAM

Objectives

The work described below constitutes the initial series of tests in a general experimental program to determine the fatigue properties of plain concrete. The objectives of these tests were to

(a) solve the practical problems associated with the fatigue testing of small concrete specimens,

(b) determine the validity of the concrete fatigue envelope shown in Fig. 2,*

(c) compare the results of high speed fatigue tests at 9000 cycles per minute with the results of fatigue tests at 500 cycles per minute.

It was hoped that the comparison of the tests at different speeds would indicate that satisfactory concrete fatigue data can be obtained from high speed tests. This is of prime importance, for high speed testing results in an enormous saving in time and makes possible a far more extensive test program. Whereas one cylinder test to one million cycles requires thirty-six hours at 500 cycles per minute, it may be completed in less than two hours using the high speed equipment.

*The fatigue envelope for concrete in Fig. 2 was originally proposed by Ros, see Reference 1.

Test Specimens

Fatigue tests were conducted on a total of thirty-four plain concrete specimens two inches in diameter and four inches long. Twenty-five specimens were tested at a speed of 9000 cycles per minute and nine at 500 cycles per minute.

Comparison static tests were carried out on twelve 2 x 4 in. cylinders and twelve 6 x 12 in. cylinders.

Materials and Mix Proportions: Concrete strength was not a desired variable in the tests, and a similar type of concrete was therefore used in all specimens. The concrete was made from high early strength (Type III) portland cement, fine Lehigh River sand and ¾ in maximum size pea gravel. The fineness moduli were found to be 3.50 and 5.87 for the sand and

Fig. 3: Preparation of Specimens

gravel, respectively. The pea gravel, which consisted primarily of sandstones and quartzites, was obtained from the banks of the Delaware River near Riegelsville in Pennsylvania. The mix proportions by weight of cement, sand and pea gravel were chosen as follows:

1.00: 2.00: 2.00

The water cement ratio was 0.50 on a weight basis, and the strength was approximately 6000 psi at 28 days.

Mixing, Placing and Curing: The concrete was mixed in a 1½ cu. ft. capacity open pan type mixer. Four batches were made successively on March 11, 12, 17 and 22. Twelve 2 x 4 in. and three 6 x 12 in. cylinders were made from each batch. The 2 x 4 in. specimens were made in four 3-gang brass molds, the concrete being placed in three well tamped layers. Standard ASTM procedure was followed with the larger cylinders.

The fifteen cylinders from each batch were initially left in their respective molds and stored in the moist room for 24 hours before removing and marking in accordance with the date and sequence of manufacture. The first 24 hours of curing was followed by a 28 day period of complete immersion in water at an average temperature of 60°F. The final stage of curing before testing involved storage in open air at approximately 70°F.

Capping: Some initial difficulties were experienced in the capping of the cylinders when mortar caps proved to be unsatisfactory for the fatigue specimens. Satisfactory performance was finally obtained from a mixture developed by the Research Laboratory of the Lehigh Portland Cement Company consisting of 6 parts sulfur, 3 parts fine bonding Ottawa sand and one part fly ash, heated to a temperature between 280°F and 290°F. Figure 3 shows the cylinder molds, ½ in. square tamping rod and capped specimens.

Load applying Agents and Instrumentation

Two types of equipment were used to apply the fatigue loading:

(a) Amsler Vibrophore apparatus, operating at 9000 cycles per minute.

(b) Amsler Hydraulic Jack and Amsler Pulsator, operating at 500 cycles per minute.

The Vibrophore is a 22,000 lb. maximum capacity machine which can apply load at

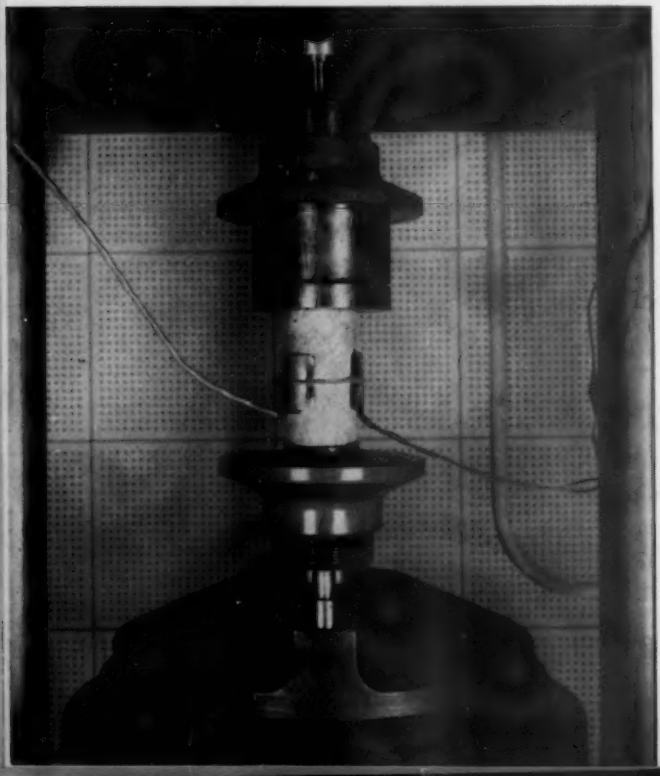


Fig. 4: Specimen in Vibrophore apparatus

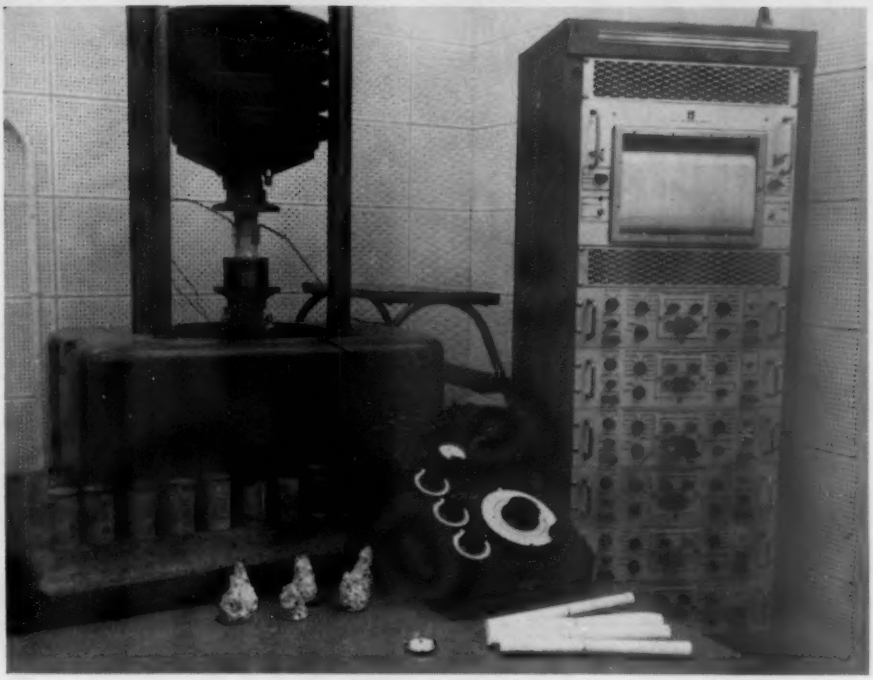


Fig. 5: 9,000 CPM test set-up

frequencies between 3,600 and 18,000 cpm. The machine operates on a resonance principle, wherein the loading rate coincides with the natural frequency of the vibrating elements of the system. The mean load is applied mechanically to the specimen and the amplitude of the load cycle is adjusted and maintained by a photo-electric regulating device. The operation of the machine is stopped at the moment of fracture of the

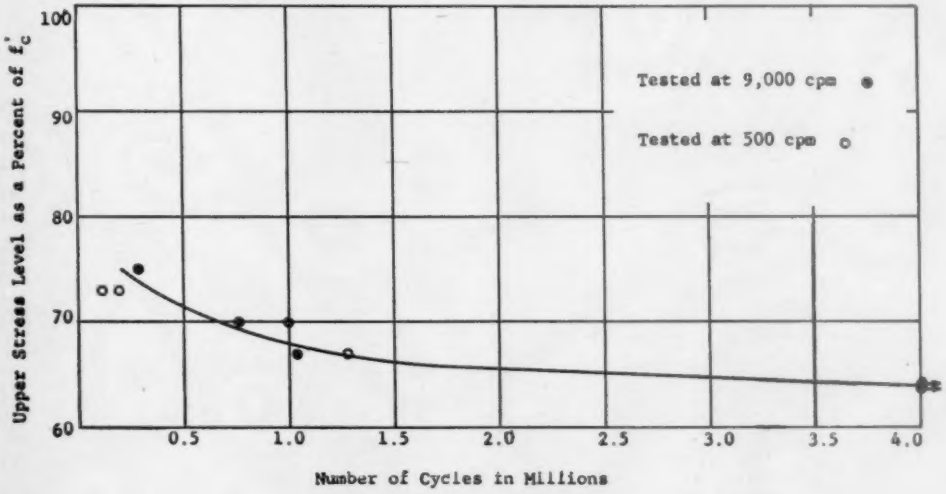


Fig. 6- S-N CURVE FOR 10 PERCENT MINIMUM STRESS LEVEL

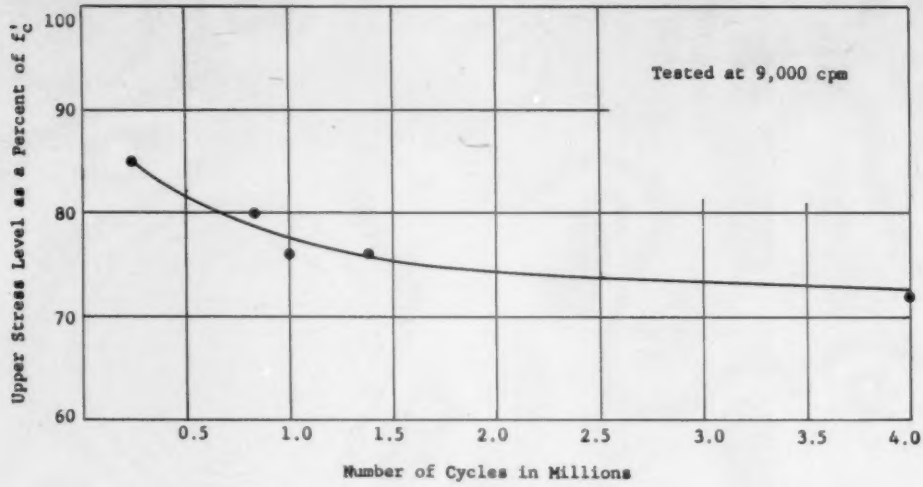


Fig. 7 - S-N CURVE FOR 30 PERCENT MINIMUM STRESS LEVEL

specimen by a relay. By adjusting the sensitivity of the relay the machine may be set to stop either when the first crack appears or when complete failure occurs. Photographs of the test apparatus may be seen in Figs. 4 and 5.

For the 500 cpm tests the specimen was centered on a solid steel base plate beneath the head of a hydraulic Amsler Jack of 22,000 lbs. capacity. The 500 cpm load cycle was achieved by means of an Amsler Pulsator connected to the jack.

Temperature gaging and recording apparatus was used to measure the variation in the surface temperature of the fatigue specimens during testing. Two RDF Stikon* type BN-1 fine nickel, wire resistance thermometer gages were attached directly to the exposed concrete surface. The gage leads were connected to a Universal type amplifier and a continuous record of temperature was obtained by feeding the outputs from the

*Manufactured by A. C. Ruge Associates Inc., Cambridge, Mass.

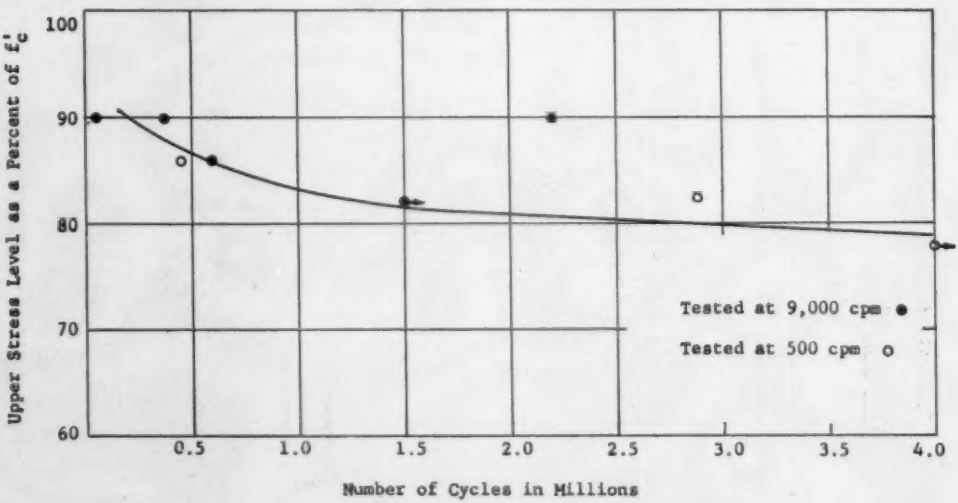


Fig. 8- S-N CURVE FOR 50 PERCENT MINIMUM STRESS LEVEL

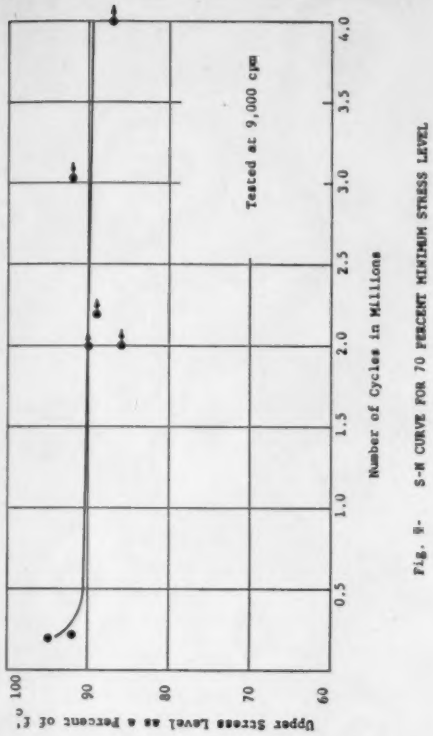


Fig. 9- S-N CURVE FOR 70 PERCENT MINIMUM STRESS LEVEL

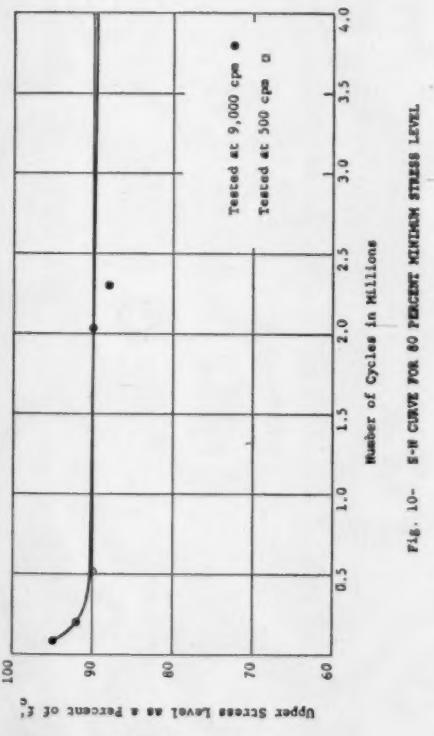


Fig. 10- S-N CURVE FOR 80 PERCENT MINIMUM STRESS LEVEL

amplifier into six channel Brush oscillograph.

The companion static tests were made using 60,000 lb. capacity and 300,000 lb. capacity universal testing machines. A number of the 2 x 4 in. static specimens were instrumented with SR4 strain gages of Type A1 to obtain stress-strain curves for the concrete.

TEST RESULTS

Fatigue Tests

Five different minimum stress levels, at 10, 30, 50, 70 and 80 percent of the ultimate strength were used in the fatigue tests. The specimens to be tested at the higher speed of 9000 cpm were divided into five groups with each group corresponding to a particular minimum stress level. The specimens in any one group were tested at the same minimum stress level, but different maximum stress levels were chosen to produce failures after a number of cycles varying between 250,000 and 4,000,000.

An S-N curve was plotted for each of the five groups by expressing the upper stress level as ordinate against the number of cycles required to cause failure as abscissa. A discrete minimum stress level is of course associated with each of the resulting curves, shown in Figs. 6 through 10.

Fatigue failure was not obtained in a number of the specimens, even after prolonged cycling. In such cases it was the procedure to conduct a static test to failure. These cylinders are distinguished in the figures by a small horizontal arrow attached to the plotted point.

A total of nine 2 x 4 in. cylinders were tested at the slower speed of 500 cpm and were duplications of previous tests at 9000 cpm. Except for one specimen which was tested with an 80 percent minimum stress level, minimums of 10 and 50 percent were used. The results are plotted as open circles in Figs. 6, 8 and 10.

Tables 1 through 4 give data for the 34 fatigue tests. The applied stress ranges given in the third and fourth columns as percentages of f'_c, were computed on the basis of 6290, 6280, 6030 and 6000 psi in tables 1, 2, 3 and 4, respectively.

The initial and final temperature readings for all fatigue tests conducted at 9000 cpm are included in the tables. A typical variation of temperature with number of ap-

TABLE 1—FATIGUE TESTS OF CYLINDERS 3/11

Specimen Number	Age at Test days	Stress Range % of f'_c max.	Stress Range % of f'_c min.	Initial Temp. °F	Final Temp. °F	Freq. (cpm)	Cycles at End of Test	Remarks
3/11-1	34	67	10	78	175	9,000	1,038,000	F
3/11-2	34	64	10	74	151	9,000	4,112,000	N.F.
3/11-3	34	64	10	—	—	500	4,000,000	N.F.
3/11-5	34	70	10	74	175	9,000	770,000	F
3/11-6	35	76	30	78	115	9,000	1,382,000	F
3/11-7	35	80	30	78	151	9,000	830,000	F
3/11-9	35	76	30	78	108	9,000	1,000,000	F
3/11-10	36	82	50	—	—	500	2,883,000	N.F.
3/11-11	36	82	50	78	101	9,000	1,500,000	N.F.

Assumed f'_c is 6290 psi, based on static tests of 3/11-4; 3/11-8; 3/11-12

F —Failure
N.F.—No Failure

TABLE 2—FATIGUE TESTS OF CYLINDERS 3/12

Specimen Number	Age at Test days	Stress Range % of f'_c max.	Stress Range % of f'_c min.	Initial Temp. °F	Final Temp. °F	Freq. (cpm)	Cycles at End of Test	Remarks
3/12-1	39	86	70	78	94	9,000	2,000,000	N.F.
3/12-2	39	90	70	78	95	9,000	2,000,000	N.F.
3/12-3	39	86	50	80	97	9,000	600,000	F
3/12-5	39	86	50	—	—	500	451,000	F
3/12-6	39	67	10	—	—	500	1,286,000	F
3/12-7	41	92	70	78	86	9,000	3,013,000	N.F.
3/12-9	42	72	30	80	161	9,000	4,000,000	N.F.
3/12-10	43	90	50	78	156	9,000	2,200,000	F
3/12-11	43	70	10	—	—	500	1,002,000	F

Assumed f'_c is 6280 psi, based on static tests of 3/12-4; 3/12-8; and 3/12-12.

F —Failure
N.F.—No Failure

TABLE 3—FATIGUE TESTS OF CYLINDERS 3/17

Specimen Number	Age at Test days	Stress Range % of f'_c max.	min.	Initial Temp. °F	Final Temp. °F	Freq. (cpm)	Cycles at End of Test	Remarks
3/17-1	38	90	50	78	118	9,000	375,000	F
3/17-2	38	90	80	—	—	500	515,000	F
3/17-3	38	88	80	78	87	9,000	2,300,000	N.F.
3/17-6	37	92	80	78	82	9,000	200,000	F
3/17-7	37	95	80	78	80	9,000	80,000	F
3/17-9	37	75	10	80	165	9,000	292,000	F
3/17-10	38	85	30	80	112	9,000	230,000	F
3/17-11	39	90	50	79	102	9,000	50,000	F

Assumed f'_c is 6030 psi, based on static tests of 3/17-4; 3/17-8; and 3/17-12

F —Failure
N.F.—No Failure

TABLE 4—FATIGUE TESTS OF CYLINDERS 3/22

Specimen Number	Age at Test days	Stress Range % of f'_c max.	min.	Initial Temp. °F	Final Temp. °F	Freq. (cpm)	Cycles at End of Test	Remarks
3/22-1	35	95	70	78	82	9,000	190,000	F
3/22-2	35	92	70	78	85	9,000	214,000	F
3/22-3	35	78	50	—	—	500	4,000,000	N.F.
3/22-6	36	87	70	77	87	9,000	4,000,000	N.F.
3/22-7	36	89	70	78	93	9,000	2,200,000	N.F.
3/22-9	36	90	80	80	88	9,000	2,040,000	F
3/22-10	36	73	10	—	—	500	200,000	F
3/22-11	37	73	10	—	—	500	117,000	F

Assumed that f'_c is 6000 psi, based on static tests of 3/22-4; 3/22-8; 3/22-12.

F —Failure
N.F.—No Failure

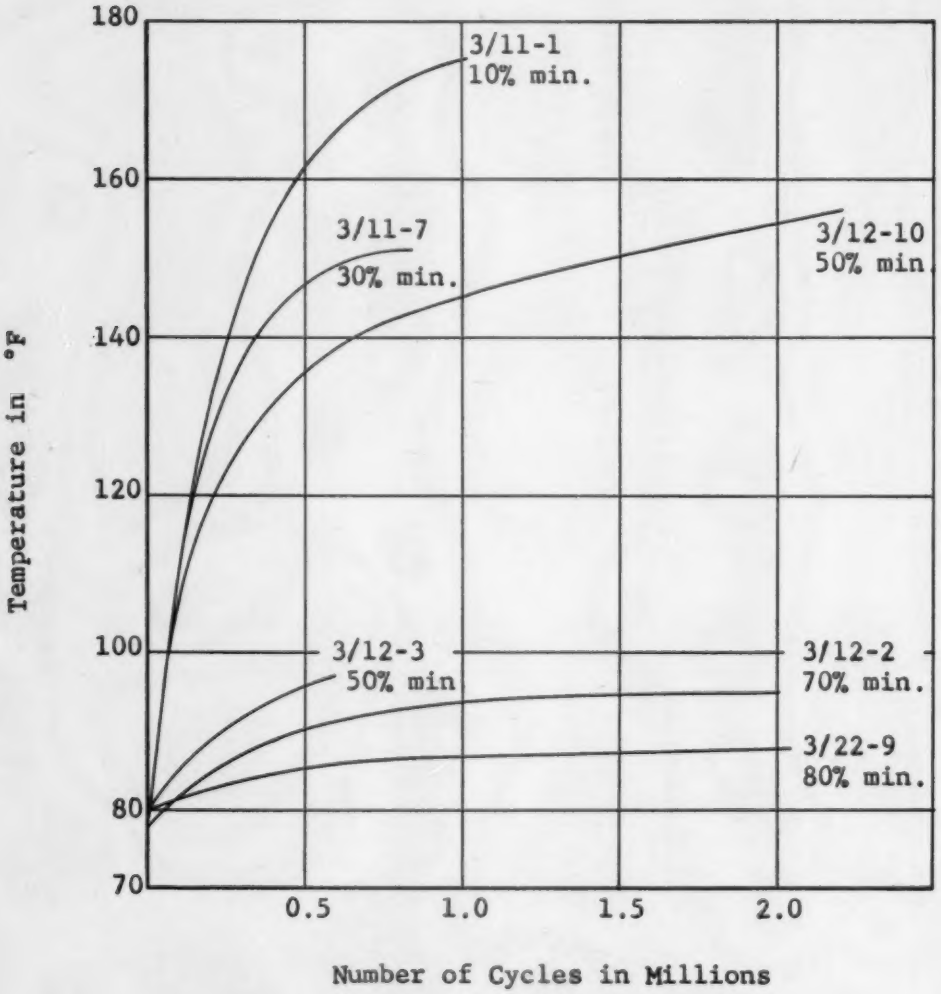


Fig. 11 - TEMPERATURE VERSUS NUMBER OF CYCLES

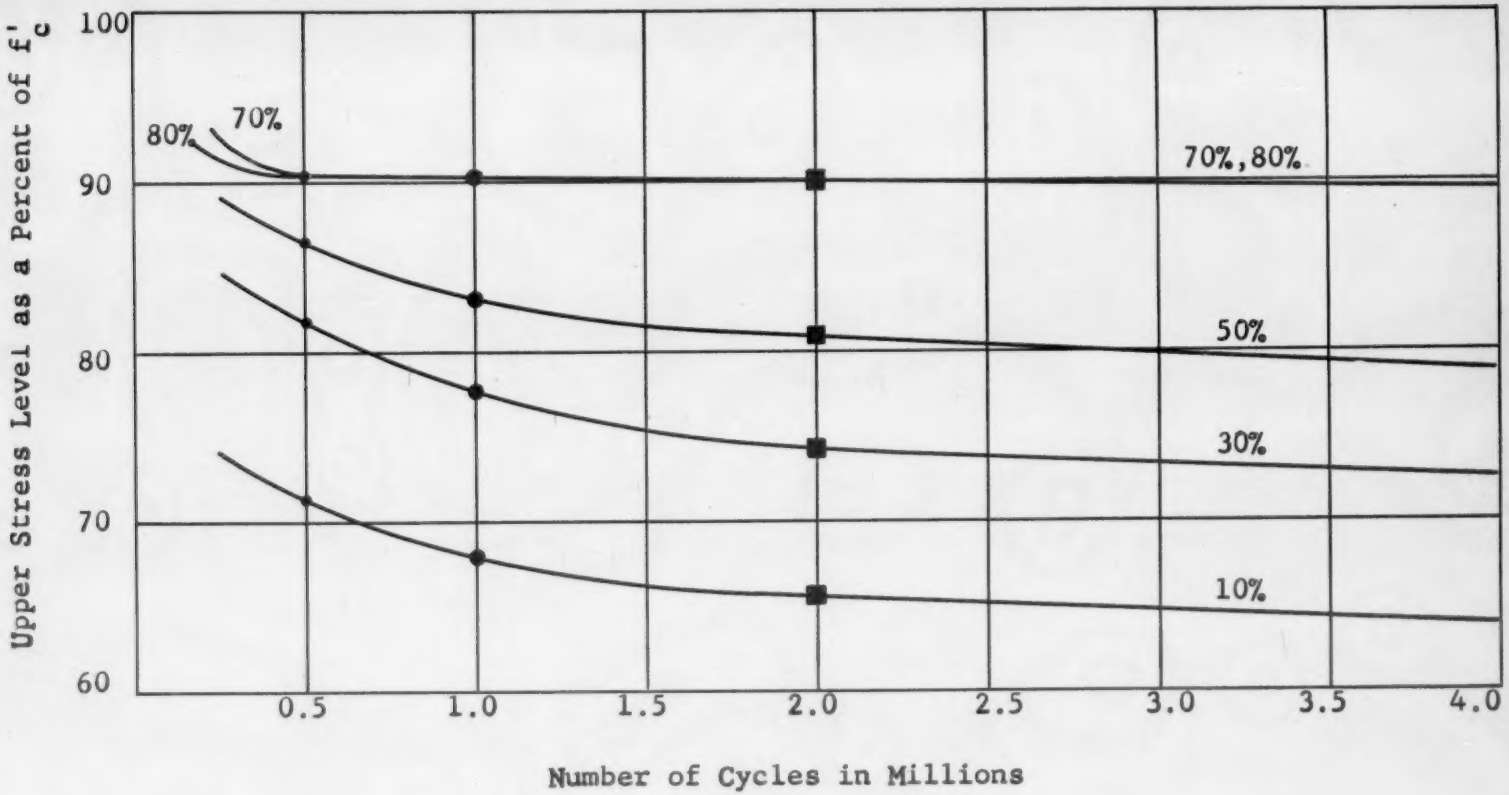


Fig. 12- S-N CURVES

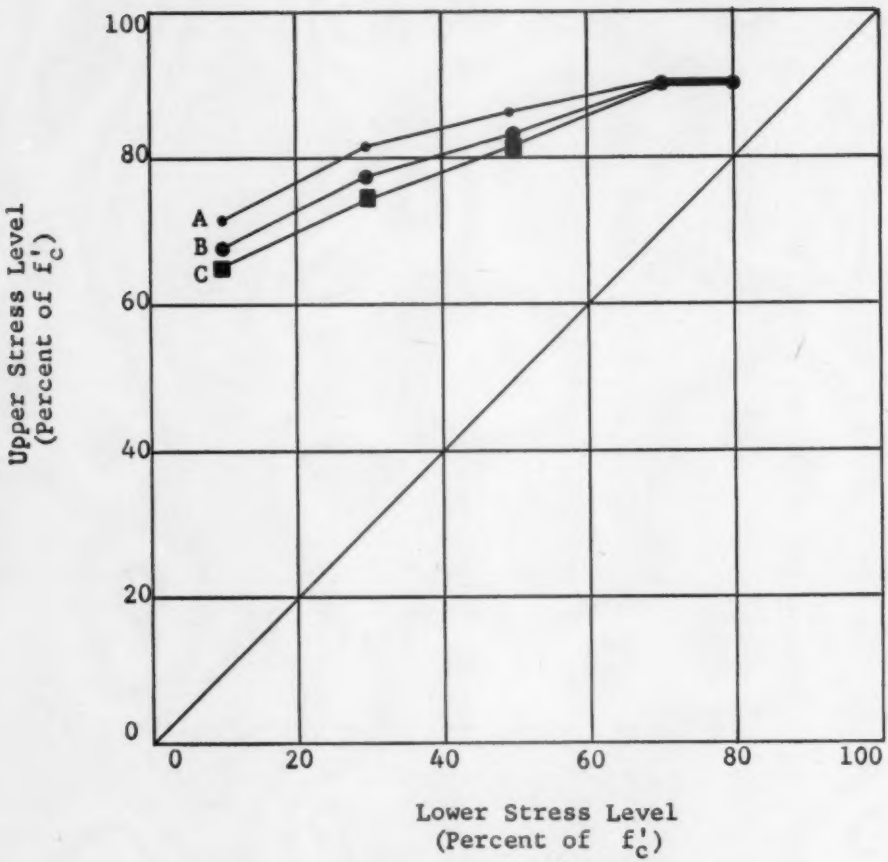


Fig. 13 - FATIGUE FAILURE ENVELOPES FOR PLAIN CONCRETE

plied cycles, for each minimum stress level, is shown in Fig. 11. No temperature increase was detected in the fatigue tests at 500 cycles per minute and instrumentation was not continued for the slower speed testing.

Derivation of Fatigue Envelopes

Fatigue failure envelopes were derived from the S-N curves contained in Figs. 6 through 10. The S-N curve, it will be remembered, indicates the number of cycles of fatigue loading required to cause failure at any given stress range. For example Fig. 6 shows that 500,000 cycles of loading between a minimum of 10 and a maximum of 72 percent of the static ultimate will induce failure, whereas 2,000,000 cycles would be required for the range from 10 to 65 percent. The S-N curves for all of the given minimum stress levels are shown in one diagram in Fig. 12. Inspection of the curves indicates the range of stresses which will cause failure at any particular number of cycles. Considering the 1,000,000 cycle level it can be seen that the following are critical stress ranges: 10-72, 30-82, 50-87, 70-90.5 and 80-90.5 percent. By replotting these figures the failure envelope for one million cycles, contained in Fig. 13, is obtained. A similar procedure established the envelopes for 2,000,000 and 500,000 cycles. If the reader wishes to review his understanding of the diagrams of Fig. 13, he is invited to check the ordinates and abscissas of the plotted points against the curves contained in Fig. 12.

Static Tests

All 6 x 12 in. cylinders were tested statically at 28 days to obtain a standard comparison strength for the concrete. The 2 x 4 in. static specimens which served as a basis for expressing fatigue strengths in terms of f'_c were also carried out in 28 days. Additional 2 x 4 in. cylinders were static tested at ages varying between 28 and 59 days to measure the increase in strength of the concrete with time. The results of all of the static tests are contained in Tables 5 and 6.

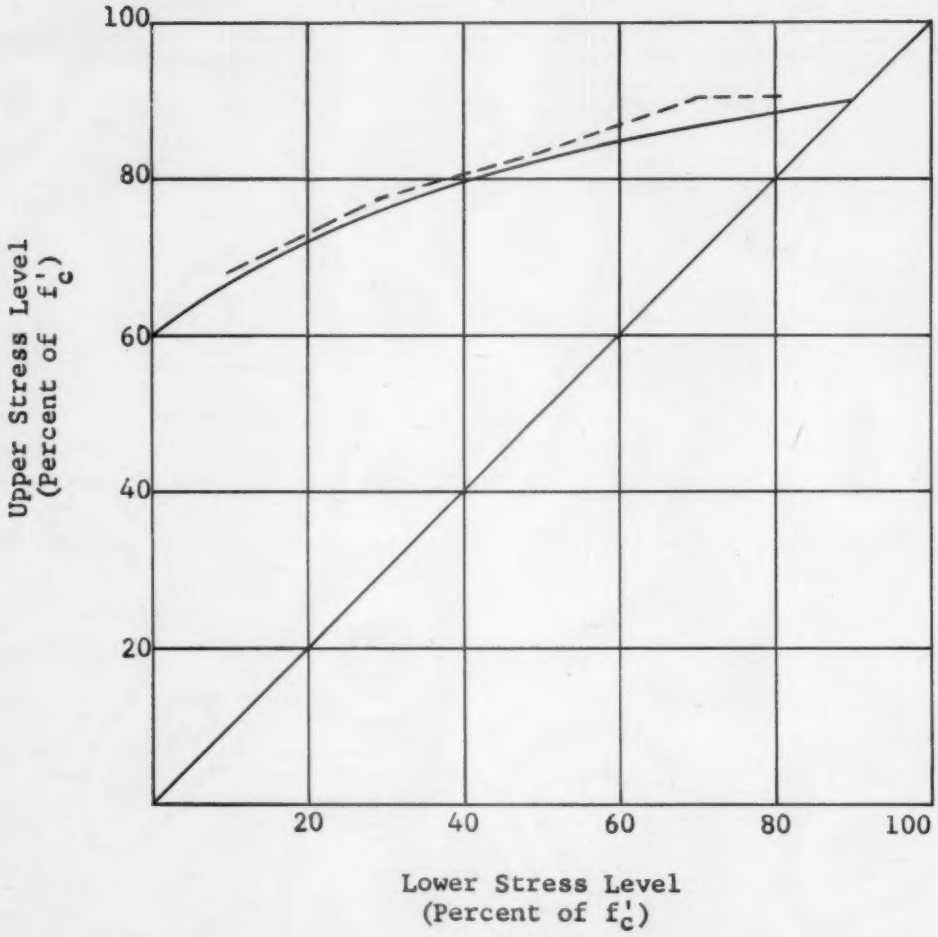
DISCUSSION OF TEST RESULTS

Fatigue Failure Envelopes

A study of the points plotted on the S-N curves of Figs. 6 through 10 indicates that

TABLE 5—STATIC TESTS ON CYLINDERS 3/11 THROUGH 3/12

Spec. No.	Size (in.)	Ult. Stress (psi)	Age at Test (days)	No. of Previous Load Cycles
3/11-4	2x4	6,180	28	0
3/11-8	2x4	6,370	28	0
3/11-12	2x4	6,310	28	0
3/11-2	2x4	6,130	59	4,112,000
3/11-3	2x4	6,200	59	4,000,000
3/11-10	2x4	4,800	59	2,888,000
3/11-11	2x4	6,440	59	1,500,000
3/11-A	6x12	5,840	28	0
3/11-B	6x12	5,940	28	0
3/11-C	6x12	6,100	28	0
3/12-4	2x4	6,240	28	0
3/12-8	2x4	6,340	28	0
3/12-12	2x4	6,270	28	0
3/12-1	2x4	6,370	58	1,500,000
3/12-2	2x4	6,370	58	2,000,000
3/12-7	2x4	6,620	58	3,013,000
3/12-9	2x4	6,250	58	4,000,000
3/12-A	6x12	5,940	28	0
3/12-B	6x12	5,800	28	0
3/12-C	6x12	5,940	28	0



Note: Dotted line obtained from tests,
Full line represents original envelope.

Fig. 14 - COMPARISON OF RESULTS WITH ENVELOPE OF FIG. 2.

TABLE 6—STATIC TESTS ON CYLINDERS 3/17 THROUGH 3/22

Spec. No.	Size (in.)	Ult. Stress (psi)	Age at Test (days)	No. of Previous Load Cycles
3/17/4	2x4	6,000	28	0
3/17/8	2x4	6,070	28	0
3/17/12	2x4	6,030	28	0
3/17/5	2x4	5,900	47	0
3/17/3	2x4	6,020	53	2,300,000
3/17-A	6x12	5,580	28	0
3/17-B	6x12	5,650	28	0
3/17-C	6x12	5,800	28	0
3/22-4	2x4	5,900	28	0
3/22-8	2x4	5,980	28	0
3/22-12	2x4	6,120	28	0
3/22-5	2x4	6,000	45	0
3/22-3	2x4	5,900	48	4,000,000
3/22-6	2x4	6,020	48	4,000,000
3/22-7	2x4	6,100	48	2,200,000
3/22-A	6x12	5,800	28	0
3/22-B	6x12	5,720	28	0
3/22-C	6x12	5,650	28	0

the scatter involved in tests at the lower minimum stress levels is, in general, less than at the higher minimum stress levels. For example, the points in Figs. 6 and 7, for 10 and 30 percent minimums, lie reasonably close to the best fit line. On the other hand the curves for 50 and 70 percent minimums, contained in Figs. 8 and 9, have been drawn through a considerable scatter of points.

In Fig. 14 a comparison is made of the results of the present tests with the original fatigue failure envelope of Fig. 2. The two curves lie close together in the lower minimum stress region but some divergence occurs in the upper region where the scatter is greater.

Temperature Effects

Although the temperature data was not entirely self-consistent, a number of observations may be made.

The temperature change during all 500 cpm tests was negligible.

The rise in temperature in the 9000 cpm tests was often considerable. A study of the initial and final temperatures listed in Tables 1 through 4 and the graphs in Fig. 11 indicate that the greatest change in temperature occurred in the tests with the lowest minimum stress level. In these tests the temperature rose from room temperature to approximately 175°F. In the tests at 80 percent minimum stress level the rise was less than 10°F.

The amount of temperature change is connected with the range of stress variation and it should be noted that the range of stress used in the tests was inversely proportional to the minimum stress level. Thus the temperature increase was generally in direct proportion to the range of stress variation. Although considerable increases in temperature did occur in a number of the specimens, no adverse effect on their strength can be seen in the test results.

Speed of Testing

The 500 cpm tests may be compared with the 9000 cpm tests in Figs. 6, 8 and 10. The unfilled circles representing 500 cpm tests are seen to be distributed above and below the S-N curves which have been drawn as the bet fit to all of the points.

Although the amount of test data is too small to derive final conclusions, no con-

sistent variation can be observed between the two rates of loading and further tests at high speed are warranted.

Mode of Failure of Fatigue Specimens

An interesting observation was made during the tests concerning the appearance of failure. Although the general orientation of fracture surfaces was similar in both static and fatigue failures, it was noticed that the failure surfaces in the fatigue specimens contained considerably fewer broken pieces of aggregate. There was apparently a strong tendency for the fatigue failure to occur in the cement sand matrix and at the matrix-aggregate interfaces rather than in the elements of aggregate.

CONCLUSIONS

It is thought that the experimental work described in this paper has resulted in the development of a satisfactory technique for high speed fatigue testing of small concrete cylinders.

On the basis of a very limited number of test results the following conclusions may be stated:

1. No effect of rate of loading on fatigue strength was observed. Although considerable changes in temperature can occur at the lower minimum stress levels, more extensive high speed fatigue testing is warranted. In conjunction with such testing a limited number of companion tests at 250 cpm or 500 cpm should be conducted.
2. The test data was substantially in agreement with the fatigue envelope for concrete as shown in Fig. 2.
3. Fatigue failure in the specimens tended to occur in the cement paste and at the surface of bonding between aggregate and paste, rather than in the aggregate itself.

REFERENCES

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2. Lane, R. E.; Ekberg, C. E., Jr.; REPEATED LOAD TESTS ON 7-WIRE PRESTRESSING STRANDS, Progress Report 21, Fritz Engineering Laboratory, Lehigh University.
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4. Knudsen, K. E.; Eney, W. J.; ENDURANCE OF A FULL-SCALE PRETENSIONED CONCRETE BEAM, **Proceedings, Highway Research Board,** P. 103, Vol. 36 (1957).
4. Ozell, A. M.; Ardamann, E.; FATIGUE TESTS OF PRETENSIONED PRESTRESSED BEAMS, **Journal of ACI,** Vol. 28, No. 4 (October 1956).
6. Inamata, S.; ON A BENDING FATIGUE TEST OF PRESTRESSED CONCRETE BEAMS, **Journal of the Japanese Society of Civil Engineers,** Vol. 37, No. 10 (October 1952).
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The authors wish to thank Mr. S. B. Helms, research engineer of the Lehigh Portland Cement Company, for his assistance with problems in connection with the capping of the specimens.

LITERATURE ON PRESTRESSED CONCRETE

By

PROF. T. Y. LIN*

Professor of Civil Engineering
University of California, Berkeley, California

(1) INTRODUCTION

Literature on prestressed concrete has now become so voluminous that it is no longer possible to compile a satisfactory and all-inclusive bibliography. However, there have been many requests from individuals interested in prestressed concrete to get some references, either at the beginner's level or for the pursuit of a specific subject. In order to meet these requirements at least partly, this list of literature is presented. While it does not give any titles of the articles, it should serve as a guide as to where to dig into the volumes and to locate the papers of one's choice.

It is hoped that readers will add information to this listing by calling to the attention of the author any important omissions. If warranted, an addendum to this listing may be printed in a future issue of this Journal.

* Director, P. C. I.

Literature on prestressed concrete can be grouped under six headings:

(A) Bibliographies
(B) Treatises
(C) Conference Proceedings
(D) Journals and Periodicals
(E) Pamphlets and Research Reports
(F) Specifications

These will be separately listed as follows:

(A) Bibliographies — Some bibliographies are given. But, as can be noticed, recent ones are lacking. This is partly because of the difficulty in keeping up with the numerous new articles. It is believed that the Cement and Concrete Association of London is still trying to keep up a list of such references. But it is questionable how complete such a bibliography can be.

1. BIBLIOGRAPHY NO. 7 "Prestressing and Precasting Concrete for Highway Bridges and Structures." The Highway Research Board, National Research Council, 2101 Constitution Avenue, Washington, D. C., 1949.
2. BIBLIOGRAPHY NO. 2 "Bibliography on Prestressed Reinforced Concrete." The Engineering Society Library, 29 W. 39th Street, New York.
3. PRESTRESSED CONCRETE (bibliography), C.A.C.A. Library Records, Cement and Concrete Association, 52 Grosvenor Gardens, London SW 1, England.
4. BIBLIOGRAPHY ON PRESTRESSED CONCRETE, ACI Committee 323, 1954.
5. BIBLIOGRAPHY ON PRESTRESSED CONCRETE, E. M. Zwoyer and I. M. Viest, Eng. Exp. Station, University of Illinois, 1952.
6. ENGINEERING INDEX

(B) Treatises—Only treatises dealing solely with the subject of prestressed concrete are listed. It will be noticed that there are now at least nine such books available in the English language.

No attempt has been made to include all books in foreign languages.

1. PRESTRESSED CONCRETE by Gustave Magnel, Concrete Publications, Ltd., 14 Dartmouth St., Westminster, London, 3rd ed., 1954.
2. PRINCIPLES AND PRACTICES OF PRESTRESSED CONCRETE by P. W. Abeles, Crosby Lockwood & Son., Ltd., 39 Thur Loe St., London S. W. 7, 1949.
3. PRESTRESSED REINFORCED CONCRETE by Kurt Billig, Knapp, Drewett & Sons., Ltd., London 1944, (later edition believed to be available).
4. PRESTRESSED CONCRETE STRUCTURES, A. E. Komendant, McGraw Hill, 1952.
5. PRESTRESSED CONCRETE, Y. Guyon, Wiley, 1953.
6. PRESTRESSED CONCRETE DESIGN AND CONSTRUCTION, F. Walley, Her Majesty's Stationery Office, London, 1953.
7. DESIGN OF PRESTRESSED CONCRETE STRUCTURES, T. Y. Lin, Wiley, New York, 1955; Spanish Edition, Editorial Continental, Mexico City, 1958.
8. THE THEORY OF PRESTRESSED CONCRETE DESIGN, H. J. Cowan, MacMillan, 1955.
9. PRESTRESSED CONCRETE, R. H. Evans and E. W. Bennett, Chapman and Hall, 1958.
10. SPANNBETON, W. Zerna, Werner-Verlag GMBH, Desseldorf, 1953.
11. SPANNBETON, Moll, Berliner Union Stuttgart, 1954.
12. VERBINDUNGEN VON STAHLBETON-FERTIGTEILEN, F. Eggeling, VEB Verlag Technik, Berlin, 1954.

13. SPANNBETON FUR DIE PRAXIS, F. Leonhardt, Wilhelm Ernst und Sohn, Berlin, 1955.
14. VORSPANNUNG MIT KONZENTRIERTEN SPANNGLIEDEN, Leonhardt-Baur, Wilhelm Ernst and Sohn, Berlin, 1956.
15. VORGESPANNTER BETON, M. Ritter and P. Lardy, Zurich, 1946.
16. BRUCKEN AUS STALHBETON UNDE SPANNBETON, E. Morsch, Konrad Wittwer Stuttgart, 1958.
17. DIN PRACTICA BETONULUI PRECOMPRIMAT, M. Halmagiu and L. Scripca, Editura Tehnica, Bucuresti, Rumania, 1957.
18. KONSTRUKCJE WSTEPNIE SPREZONE, W. Olszak, Warsaw, 1952.
19. CEMENTO ARMATO PRECOMPRESSO, C. Costelli-Guidi, Ulrico Hoepli Milano, 1956.

(C) Conference Proceedings—Many valuable papers are published in the Proceedings of several conferences on prestressed concrete. While some of these are out of print (including Proceedings, World Conference, San Francisco, 1957), many are still available. Especially recommended are the three Conference Proceedings of the International Federation for Prestressing.

1. PRESTRESSED CONCRETE, Proceedings of Conference held at Institution, Institution of Civil Engineers of London, 1949.
2. PROCEEDINGS, INTERNATIONAL CONGRESS ON PRESTRESSED CONCRETE, Ghent, Belgium, 1951.
3. FIRST U. S. CONFERENCE ON PRESTRESSED CONCRETE, PROCEEDINGS, M.I.T., 1951.
4. SYMPOSIUM ON P. C. CONTINUOUS AND FRAMED STRUCTURES, Sept. 24, 1951. CACA Publications.
5. WESTERN CONFERENCE ON PRESTRESSED CONCRETE, PROCEEDINGS, U.C.L.A., 1952.
6. CANADIAN CONFERENCE ON PRESTRESSED CONCRETE, PROCEEDINGS AND DISCUSSIONS, Univ. of Toronto, 1954.
7. UTAH PRESTRESSED CONCRETE CONFERENCE, Univ. of Utah, 1955.
8. CONFERENCE PROCEEDINGS, INTERNATIONAL FEDERATION FOR PRESTRESSING, C.A.C.A., London
   (a) First Congress, London, 1953
   (b) Second Congress, Amsterdam, 1955
   (c) Third Congress, Berlin, 1958
9. WORLD CONFERENCE ON PRESTRESSED CONCRETE, PROCEEDINGS, San Francisco, 1957.

(D) Pamphlets and Research Reports—Pamphlets on construction and design are obtainable from various organizations in the field of prestressed concrete. Research reports are published by some of the universities in this country. These are listed below. In addition, brochures describing production of standard products are available from numerous pretensioning plants, especially from Active Members of the Prestressed Concrete Institute. Post-tensioning methods and procedures are described in pamphlets by various post-tensioning systems.

1. Portland Cement Association, 33 West Grand Avenue, Chicago 10, Illinois.
2. Cement and Concrete Association, 52 Grosvenor Gardens, London SW 1, England.
3. Institute of Engineering Research, Univ. of California, Berkeley, D. of A.
4. Fritz Laboratory, Lehigh Univ., Bethlehem, Pa.
5. Talbot Lab. and Dept. of Civil Engineering, Univ. of Illinois, Urbana.
6. Dept. of Civil Engineering, Univ. of Florida, Gainesville, Florida.

(E) Journals and Periodicals—Articles on prestressed concrete now appear in almost every magazine in the field of civil, constructional, and concrete engineering. It is intended to list only the main sources of information, both in the U.S.A. and in foreign countries.

1. PRESTRESSED CONCRETE INSTITUTE, Journal (also PC Items)
2. JOURNAL ACI
3. ENGINEERING NEWS RECORD
4. CIVIL ENGINEERING
5. CONCRETE AND CONSTRUCTIONAL ENGINEERING (London)
6. CIVIL ENGINEERING (London)
7. STRUCTURAL ENGINEER (London)
8. CONCRETE CONSTRUCTION METHODS AND EQUIPMENT
9. MODERN CONCRETE
10. CONCRETE INDUSTRIES YEARBOOK
11. CONCRETE PRODUCTS
12. CONSTRUCTION REVIEW (Australia)
13. ASCE SEPARATES AND TRANSACTIONS
14. ROADS AND ROAD CONSTRUCTION
15. ROADS AND BRIDGES
16. INSTITUTION OF CIVIL ENGINEERS, JOURNAL (London)
17. MAGAZINE OF CONCRETE RESEARCH, C.A.C.A. (London)
18. MILITARY ENGINEER
19. REINFORCED CONCRETE REVIEW
20. TRAVAUX (France)
21. BAUTECHNIK (German)
22. ZEMENT (German)
23. BETON AND EISEN (German)
24. SCHWEIZERISCHE BAUZEITUNG (Switzerland)
25. PRESTRESSING-PRECONTRAINTE (Belgium)
26. DER BAUINGENIEUR (Germany)
27. BETON Y GELEZOBETON (U.S.S.R.)

(F) Specifications—Specifications for prestressed concrete have appeared in various countries. Some of these are listed below.

1. FIRST REPORT ON PRESTRESSED CONCRETE, Inst. of Structural Engineers, London, 1951.
2. INSTRUCTIONS PROVISOIRES RELATIVES A L'EMPLOI DU BETON PRECONTRAINT, Ministere des Travaux Publics, des Transports et du Tourisme, 1953.
3. INSTRUCTIONS RELATIVES AU BETON PRECONTRAINT, SECO, Bruxelles, 1954.
4. CRITERIA FOR PRESTRESSED CONCRETE BRIDGES, BPR, Wash., D. C., 1954.
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6. British Standards, Institution, Draft Code for Comment, THE STRUCTURAL USE OF PRESTRESSED CONCRETE IN BUILDINGS, Institution of Structural Engineers, 1957.
7. TENTATIVE RECOMMENDATIONS FOR PRESTRESSED CONCRETE, ACI-ASCE Joint Committee, 1958.
8. SPECIFICATIONS FOR PRETENSIONED PRESTRESSED CONCRETE, 1957 and SPECIFICATIONS FOR POST-TENSIONED PRESTRESSED CONCRETE, 1958, Prestressed Concrete Institute.

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EDITOR'S NOTE:

A novel idea introduced by the Federation Internationale de la Precontrainte (F.I.P.) makes possible an international symposium by mail, bringing here the world's experience and advice. As members of F.I.P. our prestressing knowledge is constantly being augmented through a Literature Exchange Scheme with foreign countries. Notice No. 22 just received offers an additional service — the circulation to all member groups of actual problems for solutions. The German group was first to present a problem. It was an itemized questionnaire on failure of prestressing steel. The Executive Secretary, PCI, circulated copies to selected Active, Associate and Professional Members whose replies are presented below.

We take this occasion to express our gratitude to those PCI contributors for their interest and celerity in responding, and we now open this section to the readers for comment and discussion.

The questions are repeated for each reply received in order that they be evaluated in terms of the individual's experience and the merging of replies.

Following is a summary of PCI answers to the F.I.P. questionnaire. Notice that several replies make reference to the word "heat" and even "heat-treated," but it appears that the F.I.P. meaning of "heat treatment" is probably as defined by one U. S. steel supplier as follows: "It is assumed that 'heat-treated' refers to steel in which the high strength is a result of heating and quenching and has no reference to stress-relieving." Under this definition, it is possible that very little "heat-treated" steel has been used in Prestressed Concrete in the United States.

Note also that stress-corrosion has been blamed for steel failures only in circular structures in the U.S. — none of the answers link stress corrosion to steel failures in linear structures. It is quite likely that many of the comments cover the same tank structure, and there is not complete agreement that stress corrosion was the cause for these failures.

Two steel suppliers did not answer the specific questions but made general comments which indicated agreement with the findings of Walter O. Everling's well known study. For example, Karl H. Middendorf, a recognized authority on prestressing steel, comments as follows: "As to group 2 items, the only research I have knowledge of is that made under the direction of Walter O. Everling, Director of Research, American Steel & Wire, and reported at the American Iron and Steel Institute in 1954. His conclusions: stress corrosion failures occur only in the presence of Nitrate solutions; oil-tempered wire substantially more

sensitive than hard drawn; failure occurs only when protection against corrosion breaks down."

Many of the prestressing steel suppliers in the U. S. feel that cold-drawn, stress relieved steel is superior to heat-treated (oil-quenched) steel with regard to stress corrosion.

—M.P.K.

A. PRESTRESSING STEEL SUPPLIER

QUESTIONNAIRE

1. Have serious failures of prestressing steel been observed in your country during the past few years?
One.

1.1 In which structure(s) did the failures occur?
Water storage tank.

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)
During tensioning.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?
No.

1.23 Has failure ever been noticed after grouting?
No.

1.3 How were the failures observed? What drew attention to the failures?
Wire breaks during tensioning.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?
A large percentage of the tendons used on this project.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?
Cold drawn stress-relieved wire.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?
Failure was due to very heavy rust as a result of leaving the wire exposed to the weather for several months. The rust decreased the cross section of the wire and greatly increased the friction in the enclosure.

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?
See No. 1.7.

1.9 Have you any further information to give?
No.

2. 2.1 Have stress-corrosion failures been observed in heat-treated steel?
This and subsequent questions mention heat-treated steel. It is assumed that "heat-treated" refers to steel in which the high strength is a result of heating and quenching and has no reference to stress-relieving. Heat-treated steels are not commonly used for tensioning elements in the U.S.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?
No stress-corrosion failures have been observed in cold drawn stress-relieved wire.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?

Section 304.2 of TENTATIVE RECOMMMENDATION FOR PRESTRESSED CONCRETE says, "High tensile strength wire produced by oil tempering process is not recommended for use in prestressed concrete."

3. 3.1 What measures have been taken (or suggested) in your country to prevent such failures?

It has been found that stress-relieving of cold-drawn wire greatly increases the resistance of the wire to stress-corrosion.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?

Yes, but only laboratory test data.

4. Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?

Yes.

5. Have the steel technologists in your country found a method of preventing stress-corrosion? If so, is it satisfactory?

It cannot be entirely prevented but stress-relieved wire is much less susceptible.

B. HIGHWAY OFFICIAL

QUESTIONNAIRE

1. Have serious failures of prestressing steel been observed in your country during the past few years?

No.

1.1 In which structure(s) did the failures occur?

Bridges.

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)

During.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?

Variable.

1.23 Has failure ever been noticed after grouting?

No.

1.3 How were the failures observed? What drew attention to the failures?

Parting of the tendons during stressing.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?

Not more than 1/10 of 1%.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?

Cold drawn and stress-relieved 1" and 1⅛" rod and 7- wire strand.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?

No stress corrosion. Stress raisers caused by threaded anchors on rods. Welds on wires.

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?

No.

1.9 Have you any further information to give?

No.

2. 2.1 Have stress-corrosion failures been observed in heat-treated steel?

No.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?

No.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?

No information.

3. 3.1 What measures have been taken (or suggested) in your country to prevent such failures?

Threading rod anchorages not permitted.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?

No failures with wedge type anchorages. Occasional failures in wire welds seem inevitable.

4. Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?

No information.

5. Have the steel technologists in your country found a method of preventing stress-corrosion? If so, is it satisfactory?

No information.

C. CONSULTING ENGINEER

QUESTIONNAIRE

1. Have serious failures of prestressing steel been observed in your country during the past few years?

Yes.

1.1 In which structure(s) did the failures occur?

Water tank.

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)

After stressing, but prior to grouting.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?

80% of ultimate.

1.23 Has failure ever been noticed after grouting?

No.

1.3 How were the failures observed? What drew attention to the failures?

During re-stressing, wires broke.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?

Majority of cables in this tank.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?

Cold-drawn, stress-relieved. 5 mm. 250,000 psi ultimate.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?

Thought due to stress corrosion.

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?

Long delay between stressing and grouting.

1.9 Have you any further information to give?

No answer.

2. 2.1 Have stress-corrosion failures been observed in heat-treated steel?

Do not know.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?

See No. 1.7.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?

We believe oil-tempered wire is more susceptible, but have no opinion on cold-drawn or heat-treated.

3. 3.1 What measures have been taken (or suggested) in your country to prevent such failures?

Immediate grouting.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?

No failures on grouted steel.

4. Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?

Various steel companies have developed methods. Wire is looped and bathed in chemicals and time to break is noted.

5. Have the steel technologists in your country found a method of preventing stress-corrosion? If so, is it satisfactory?

No answer.

D. CONSULTING ENGINEER

QUESTIONNAIRE

1. Have serious failures of prestressing steel been observed in your country during the past few years?

Yes.

1.1 In which structure(s) did the failures occur?

Variety of types of structures.

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)

During stressing in most cases. But martensite formations caused breakage in one instance during normal handling of unstressed steel.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?

80% of ultimate or less.

1.23 Has failure ever been noticed after grouting?

No.

1.3 How were the failures observed? What drew attention to the failures?

Breakage of steel.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?

In proportion to total use, percentage of failures is very small and in most cases are due to negligence.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?

Heat treated, cold rolled.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?

One project had failures that are considered caused by stress corrosion. Other failures have been at welds made prior to drawing during manufacture. Standard procedure calls for removing these weld areas at the completion of the manufacturing process, but occasionally one seems to be overlooked. Other failures have been caused by careless handling, i.e., nicking the steel or subjecting it to high temperatures (burning torches, welding slag, etc. and air cooling, thereby forming martensite).

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?

See No. 1.7

1.9 Have you any further information to give?

See No. 1.7

2. 2.1 Have stress-corrosion failures been observed in heat-treated steel?

No answer.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?

No answer.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?

No answer.

3. 3.1 What measures have been taken (or suggested) in your country to prevent such failures?

No answer.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?

No answer.

4. Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?

No answer.

5. Have the steel technologists in your country found a method of preventing stress-corrosion? If so, is it satisfactory?

No answer.

E. PRESTRESSING STEEL SUPPLIER

QUESTIONNAIRE

1. Have serious failures of prestressing steel been observed in your country during the past few years?

Yes, during the past five years.

1.1 In which structure(s) did the failures occur?

Bridges and tanks.

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)

During and after prestressing.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?

Some failed at 116,000 psi, some at 133,000 psi, and some are unknown.

1.23 Has failure ever been noticed after grouting?

Failures have been noted after grouting.

1.3 How were the failures observed? What drew attention to the failures?

Failure occurred when wire broke during prestressing, when structures collapsed, and when bars broke during and after stressing operations.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?

In the case of the tanks there was complete failure of the steel. In the case of bridges, only .009% failed.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?

The steel used in the tanks was cold drawn wire, treatment is unknown by us. Diameter in one case was .162 in. strength unknown. In the case of bars, diameter 1⅛ in.;

tensile strength min. 145,000 psi. One was heat treated; one was cold drawn with mechanical stress relief.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?

In the case of wires, stress corrosion appeared to be the cause of failure. In the case of bars, surface defects and lack of ductility were the causes.

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?

In the case of the tanks the wire was exposed to moisture and salt air, and there was some delay in grouting after prestressing. No failures in bars were noted due to stress corrosion.

1.9 Have you any further information to give?

Failures in bars may be due to:

a. Surface defects (rolled in scale, slivers, laps, etc.) acting as notches or stress raisers concentrating stresses at the defect and eventual failure;

b. Cold working bars to increase tensile properties followed by stress relieving (to increase elongation) by heating in the blue brittle range;

c. Internal defects not discovered by inspection such as secondary pipe, excessive segregation, or large inclusions.

2. 2.1 Have stress-corrosion failures been observed in heat-treated steel?

Cold drawn or cold worked bars NOT stress relieved either by HEAT or MECHANICAL STRESS RELIEVING are more subject to stress corrosion than bars which have been stress relieved. The higher the residual stresses left in steel, the more it is subject to corrosion.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?

See No. 2.1.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?

See No. 2.1.

3. 3.1 What measures have been taken (or suggested) in your country to prevent such failures?

No examples of stress corrosion on bars have been brought to our attention. Due to lower ratio of surface to volume on bars as compared to wire, chances of stress corrosion on bars greatly minimized.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?

Not known.

4. Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?

No, except the fundamental knowledge of what causes stress corrosion.

5. Have the steel technologists in your country found a method of preventing stress-corrosion? If so, is it satisfactory?

Stress corrosion can be prevented by:

a. Barrier protection. (paint)

b. Cathodic protection (zinc, cadmium, or aluminum hot dip or plating).

c. Eliminating corrosive conditions surrounding the steel.

F. PRESTRESSING STEEL SUPPLIER

1. Have serious failures of prestressing steel been observed in your country during the past few years?

Occasional failures, 3 or 4 years ago.

1.1 In which structure(s) did the failures occur?

Bridge structures.

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)

In one project during stressing. In another project during stressing and within one hour after stressing.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?

Approximately .7 of ultimate.

1.23 Has failure ever been noticed after grouting?

No failures have been noticed after grouting.

1.3 How were the failures observed? What drew attention to the failures?

Any failure during stressing is immediately and explosively observed by the jacking crew. The failure after tensioning is noted by the presence of a bar projecting from the structure.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?

Less than 1%.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?

Steel used was AISI 9260, cold worked by stretching 1⅛" dia. and 1" dia. bars. Minimum guaranteed ultimate strength of 145,000 psi.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?

Failures were not due to stress corrosion. Failures were due to lack of ductility in the steel and angularity at anchorage.

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?

These conditions were not present in the case of the two failures mentioned above. They have been present in other structures completed, but no failures were noted.

1.9 Have you any further information to give?

No stress corrosion failures have been observed with the steel in question during the five years in which it has been sold and used in the U. S. Failures noted have been due to lack of ductility in the steel, anchorage problems or careless or improper handling. In the last two to three years, all but the last of these reasons for failure have been completely eliminated. A slight change in the chemical composition of the steel to attain a more ductile alloy coupled with the development and use of the stress relieving furnace has eliminated the problem of ductility. The development of the wedge anchor for bar prestressing has eliminated anchor failures. Occasional failures still result from careless handling despite efforts to educate manufacturers and product users. Principal failures of this type start from martensitic deposits on the steel caused by contact with welding spark.

2. 2.1 Have stress-corrosion failures been observed in heat-treated steel?

No comment.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?

No comment.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?

No comment.

3. 3.1 What measures have been taken (or suggested) in your country to prevent such failures?

a. The control of corrosive atmosphere; the use of calcium chloride, or for that matter, any chloride additive to prestressed concrete or the use of grouting materials containing chlorides are generally prohibited by U. S. specifications. Some states prohibit the placement of steel wires (although not bars) prior to steam curing.

b. The control of physical causes, most states require steel to be clean and free from rust prior to use. This generally requires special preparation by the manufacturer to insure that the steel can be shipped and stored satisfactorily and still be in this condition when needed. We wrap our material in waterproof, reinforced Kraft paper to prevent corrosion.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?

The fact that there have been no corrosion failures with the material in question would indicate that it is either not susceptible to stress corrosion because of its large diameter, or for other reasons, or that the measures taken have been effective to prevent corrosion.

4. Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?

No comment.

5. Have the steel technologists in your country found a method of preventing stress-corrosion? If so, is it satisfactory?

No comment.

G. CONSULTING ENGINEER

QUESTIONNAIRE

1. Have serious failures of prestressing steel been observed in your country during the past few years?

We have had serious failures of prestressing steel in some prestressed concrete tanks. To my knowledge, no corrosion has been reported for linear structures.

1.1 In which structure(s) did the failures occur?

See No. 1

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)

In one instance, using a post-tensioned wire system, the failure occurred during construction. In wire wound tanks, the failures occurred from one to five years after completion.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?

For the wire wound tanks, the stress range is probably between 115,000 psi and 150,000 psi for 210,000 psi ultimate wire.

1.23 Has failure ever been noticed after grouting?

For wire wound tanks, the prestressing steel is protected by pneumatic mortar. Failures have occurred only after the mortar has been applied.

1.3 How were the failures observed? What drew attention to the failures?

Failures were observed by spalling the mortar covercoat, and vertical cracks with subsequent leakage.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?

The percentage of wire that failed is extremely small considering the extent of the wire winding application. However, where the conditions which are conducive to corrosion exist, the corrosion is extensive.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?

For the post-tensioned wire system, stress relieved wire was used. For the wire wound tanks, cold drawn wire which is again cold drawn in the field to apply the stress is used.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?

In the case of the post-tensioned wire, it is thought to be stress corrosion. In wire wound tanks, the corrosion may have been stress corrosion where wires were deflected around openings. Generally though, the several instances of corrosion have been apparently caused by poor workmanship in covering the wires with pneumatic mortar. In chemical process tanks such as digester or brine tanks, the corrosion was generally caused by poor covercoat and/or leakage of the contained material. In a digester tank, rather high stray currents were noted, and the corrosion was apparently electrolytic corrosion which could only occur in a wet medium.

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?

Yes, definite causes, such as:

a. Leakage of stored material.

b. Stray electric currents.

c. Penetration of corrosive liquids.

1.9 Have you any further information to give?

Where wire wound prestressing is used, and the wires are completely encased in a rich cement mortar, the performance has generally been excellent.

2. 2.1 Have stress-corrosion failures been observed in heat-treated steel?

We have no experience in heat-treated steel.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?

It is believed that some cases of corrosion were stress corrosion.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?

Yes, two American steel companies have metallurgists who feel that heat treated (oil quenched) wire is many times more susceptible to stress corrosion than is stress relieved wire.

3. 3.1 What measures have been taken (or suggested) in your country to prevent such failures?

The thinking is that to avoid corrosion, the following should be adhered to:

a. Prestressing steel should be completely encased in a rich mortar.

b. Grout should preferably have the same cement as the parent concrete.

c. Calcium chloride should not be used.

d. Sharp changes in angles should be avoided.

e. It is preferable to avoid the use of oil quenched wire. It is felt that this recommendation may be on weak grounds. However, I prefer to comply with it until I can be definitely assured that oil quenched (heat treated) wire is satisfactory.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?

To my best knowledge, there has not been any wire failures after completion of the structures, when stress relieved wire is used.

4. Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?

Not to my knowledge.

5. Have the steel technologists in your country found a method of preventing stress-corrosion? If so, is it satisfactory?

I don't believe that any steel technologist has found a method of preventing stress corrosion.

H. CONSULTING ENGINEER

QUESTIONNAIRE

1. Have serious failures of prestressing steel been observed in your country during the past few years?

We have served as prestressed concrete consultants to approximately 25 plants in the United States during the past 8 years and we have not experienced a failure of a prestressed product due to failure of the prestressing strands. We have observed, in one case, a continued deflection of slabs over a period of about one year's time, resulting in serious cracking of the slabs. These slabs were roof slabs over the operating floor of a creamery. The cracks which finally formed were about 6″ on centers and extended 4″ or 5″ into a 14″ leg. The slabs deflected approximately 3″ on a 40′ span with a 13′ cantilever. One design of the slabs was checked and we found that it was correct and could offer no solution to the deflection and cracking. The slabs were manufactured, using calcium chloride and the cleaning process in the creamery involved the use of chlorine and steam, and it was our first thought that the action of the chlorine gas combined with steam may have promoted stress corrosion when combined with possible action under these conditions with the calcium chloride. A section of one of the legs of the channel slabs was removed to determine whether the strand was deflected to its proper position and if the number of strands and their size corresponded to the plans. It was found that the manufacturer had complied with these requirements. The short section of strand available for inspection, after removing a portion of the leg, did not show serious evidence of stress corrosion or action by calcium chloride. We are, therefore, still unable to explain the reason for the deflection and cracking. The slabs are still in place and still unrepaired, and apparently the owner has decided to leave existing conditions as they are. No further investigation was made and none is intended, so far as I know. The slabs have been in place approximately 3 years.

1.1 In which structure(s) did the failures occur?

See No. 1.

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)

See No. 1.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?

See No. 1.

1.23 Has failure ever been noticed after grouting?

See No. 1.

1.3 How were the failures observed? What drew attention to the failures?

See No. 1.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?

See No. 1.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?

See No. 1.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?

We are not in a position to comment on these items.

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?

No answer.

1.9 Have you any further information to give?

No answer.

2. **2.1 Have stress-corrosion failures been observed in heat-treated steel?**

No answer.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?

No answer.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?

No answer.

3. **3.1 What measures have been taken (or suggested) in your country to prevent such failures?**

No answer.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?

No answer.

4. **Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?**

No answer.

5. **Have the steel technologists in your country found a method of preventing stress-corrosion? If so, is it satisfactory?**

No answer.

I. PRESTRESSED CONCRETE PRODUCER

QUESTIONNAIRE

1. Have serious failures of prestressing steel been observed in your country during the past few years?

a. One major. b. A few minor.

1.1 In which structure(s) did the failures occur?

a. Prestressed tank.

b. Bridge girders.

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)

a. During and after stressing.

b. One month after grouting.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?

a. About 80% of ultimate.

b. About 60% of ultimate.

1.23 Has failure ever been noticed after grouting?

b. Yes, on one bridge girder job.

1.3 How were the failures observed? What drew attention to the failures?

a. Breaking of wires, spalling of concrete.

b. On bridge girder job after grouting, noticed bars projecting from end.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?

a. 50%.

b. 10%.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?

a. Cold drawn.

b. Heat treated bars.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?

a. Yes, stress corrosion and corrosion.

b. No.

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?

a. Yes, long delay, wires left several months in tubes in partially stressed condition.

1.9 Have you any further information to give?

a. Due to excessive friction, wire was overstressed in jacking.

b. No.

2. 2.1 Have stress-corrosion failures been observed in heat-treated steel?

No answer.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?

No answer.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?

No answer.

3. **3.1 What measures have been taken (or suggested) in your country to prevent such failures?**
Use of a vaporizing powder VPI. No $CaCl_2$ in concrete.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?
Yes, wire was removed after 2 months in tube with VPI; appeared free from corrosion.

4. **Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?**
No answer.

5. **Have the steel technologists in your country found a method of preventing stress-corrosion? If so, is it satisfactory?**
No answer.

J. CONSULTING ENGINEER

QUESTIONNAIRE

1. **Have serious failures of prestressing steel been observed in your country during the past few years?**
Yes.

1.1 In which structure(s) did the failures occur?
Water and sewerage tanks.

1.21 At what stage of construction did the failures occur? (Before, during, or after prestressing?)
After prestressing.

1.22 Is the approximate steel-stress known (in relation to the ultimate strength) at which failure occurred?
About 130,000 to 150,000 psi.

1.23 Has failure ever been noticed after grouting?
Yes, after guniting.

1.3 How were the failures observed? What drew attention to the failures?
Rust lines seeping out of gunite covers.

1.4 What percentage of the total quantity of prestressing steel used in each construction failed?
Several tanks about 20 to 100 tons each.

1.5 What types of steel were used? (Heat-treated? cold-drawn?) diameter and strength?
Cold Drawn.

1.7 Is it thought that the failures were due to stress-corrosion or other causes?
Stress corrosion.

1.8 Were there present, in any case of a failure, conditions likely to increase the susceptibility of the steel to corrosion (climate, long delay between prestressing and grouting, careless storage of steel, etc.)?
One case was long delay before grouting.

1.9 Have you any further information to give?
No answer.

2. **2.1 Have stress-corrosion failures been observed in heat-treated steel?**
No.

2.2 Have stress-corrosion failures been observed in cold-drawn steel?
Yes.

2.3 Have your country's experts reason to believe that one special type (e.g. cold-drawn or heat-treated) or even one particular make of steel is more susceptible to corrosion than another?
No.

3. 3.1 What measures have been taken (or suggested) in your country to prevent such failures?
None as yet, because most people don't know about it.

3.2 Is there any experience available as to the effectiveness of the measures suggested or adopted?
No answer.

4. Has a method been developed for testing the susceptibility of prestressing steel to stress corrosion?
Yes.

5. Have the steel technologists in your country found a method of preventing stress corrosion? If so, is it satisfactory?
No.

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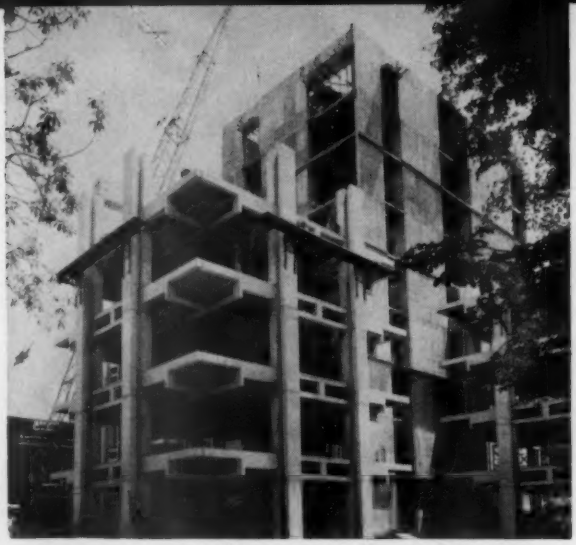

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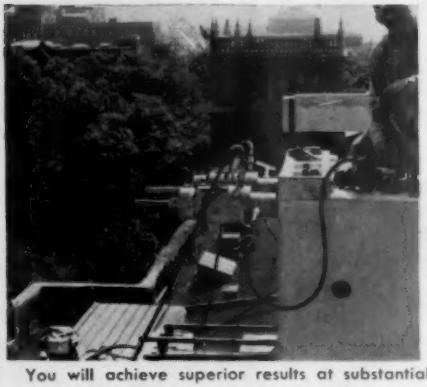

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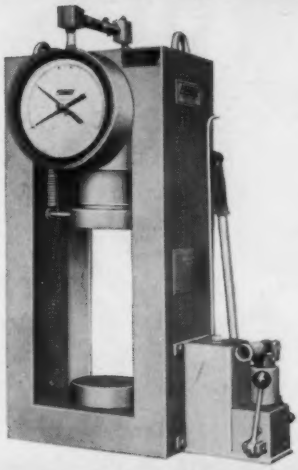

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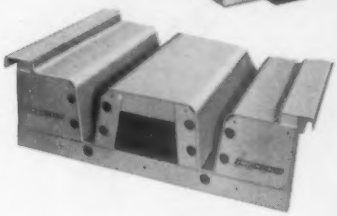

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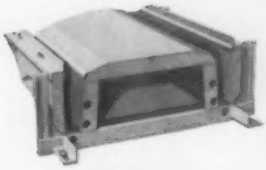

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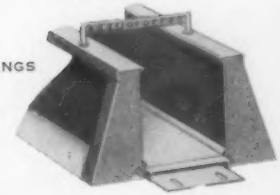

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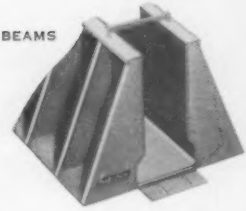

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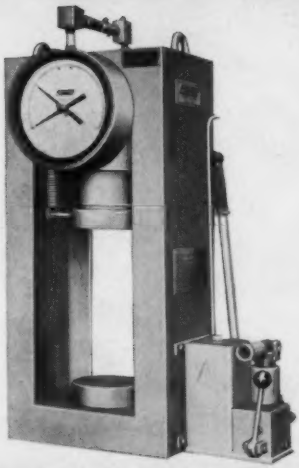

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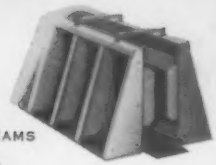

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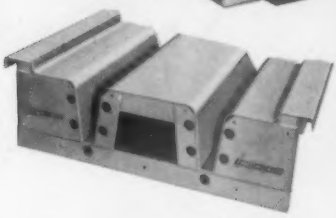

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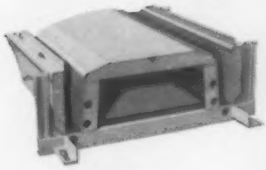

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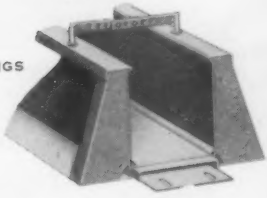

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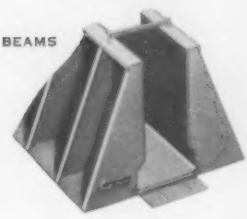

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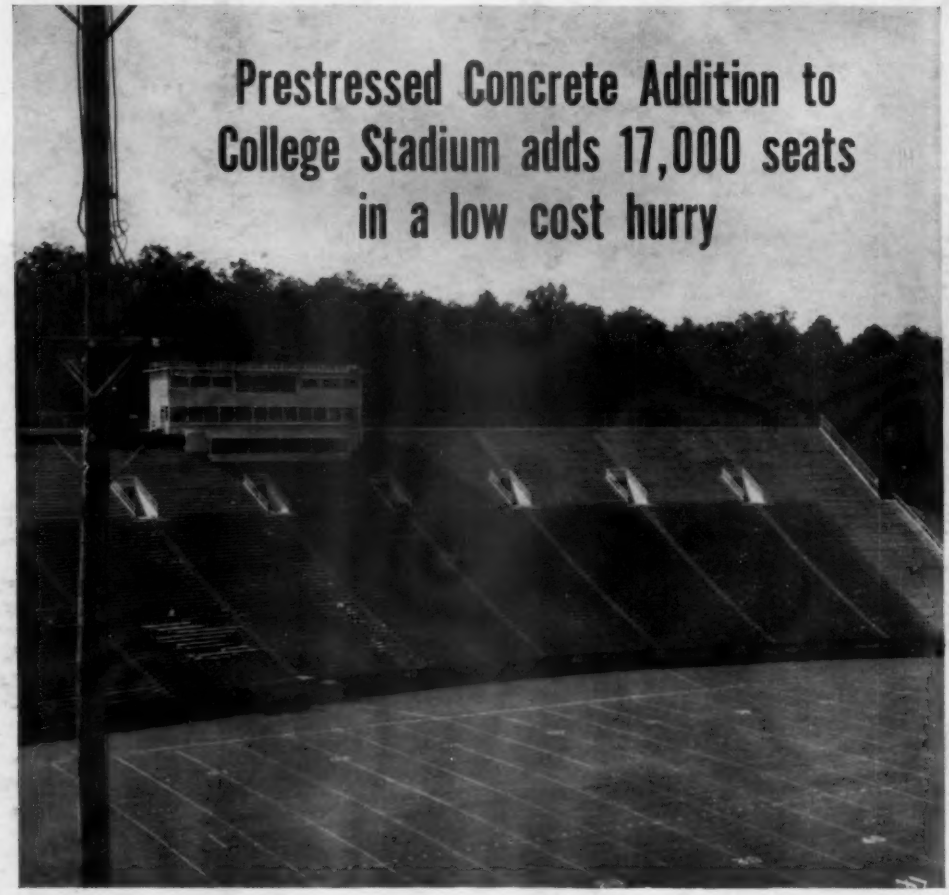

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